CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

FEBRUARY, 1950. **Vol. XLV, No. 2**

FORTY-FIFTH YEAR OF PUBLICATION

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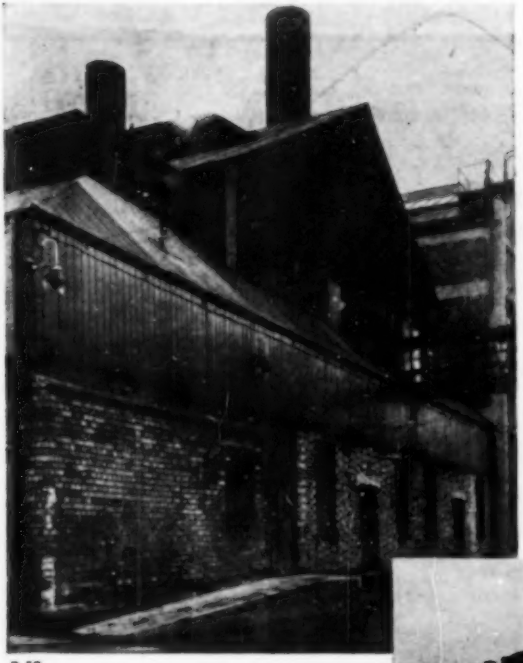

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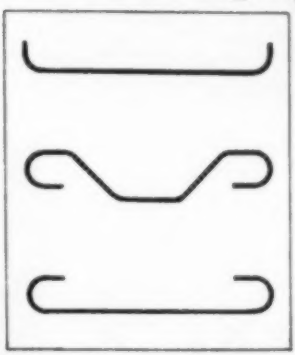

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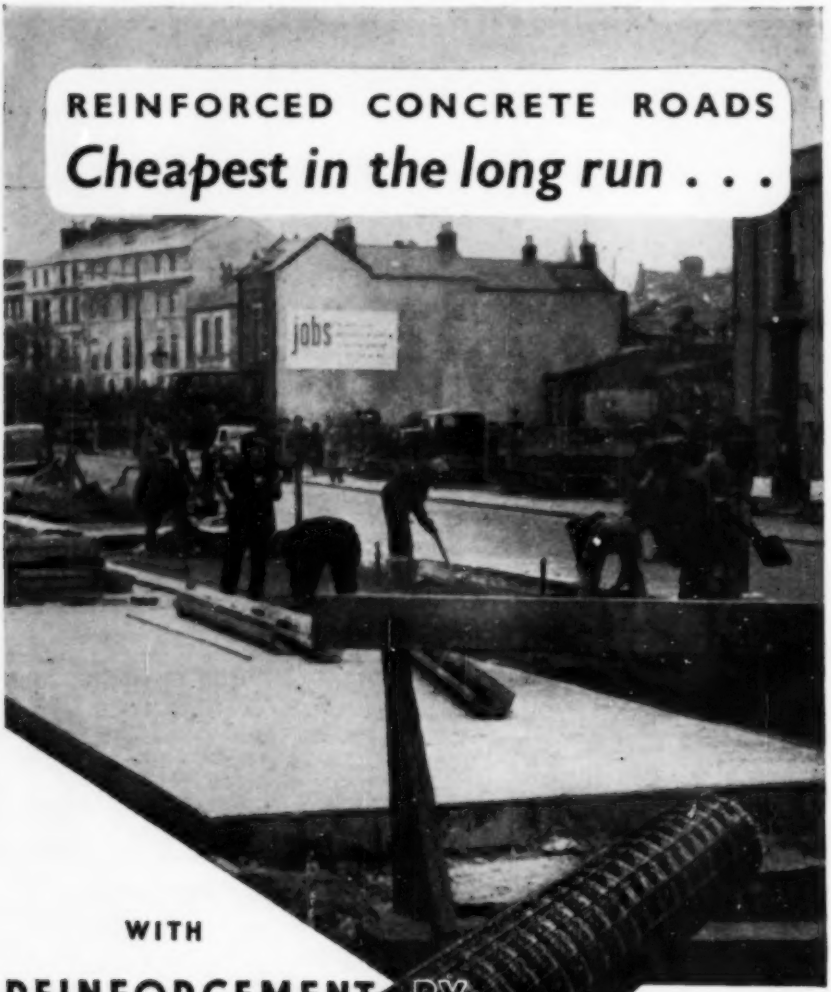

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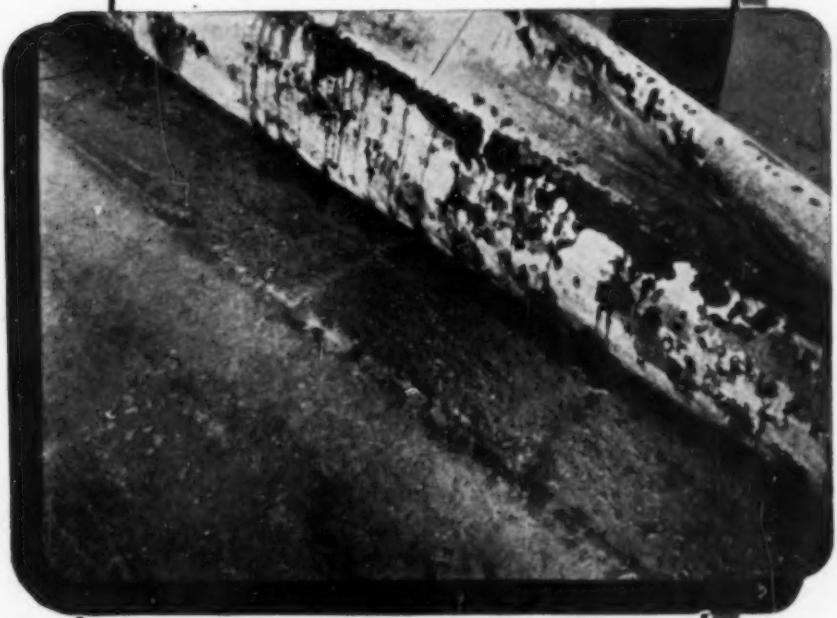

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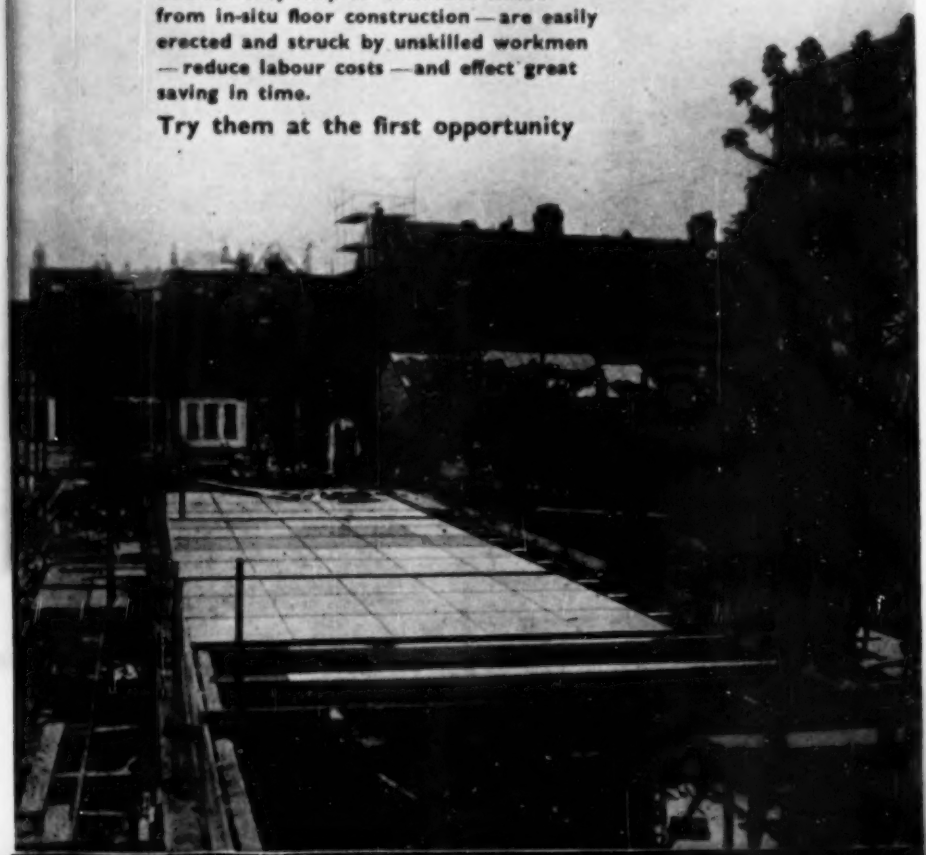

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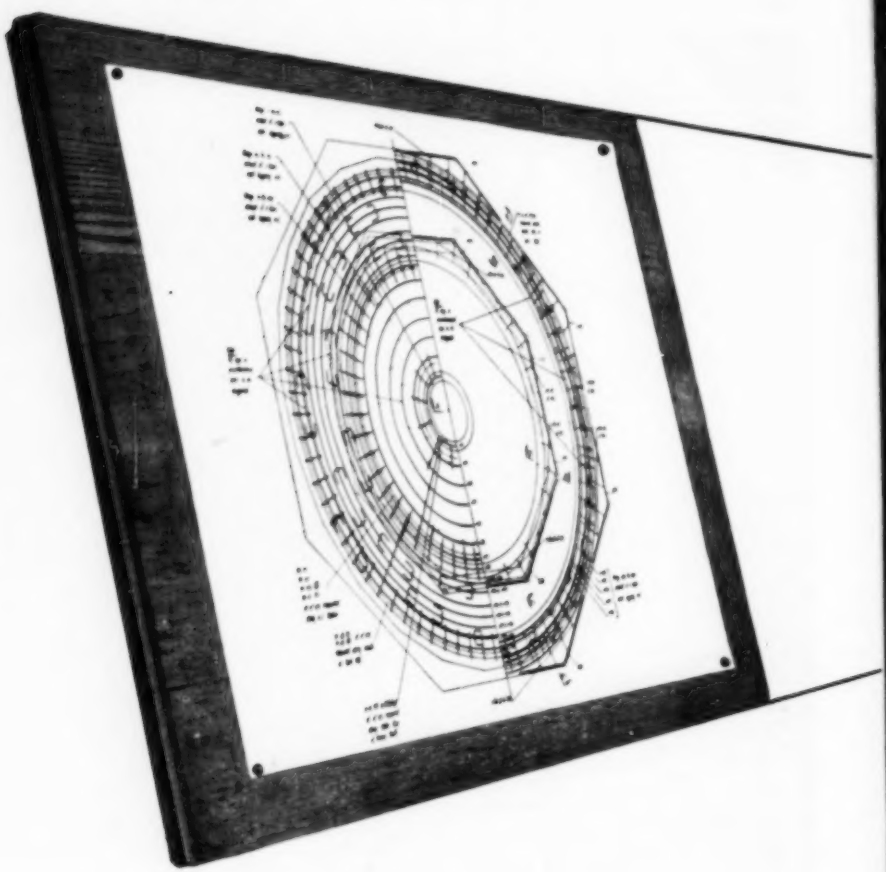

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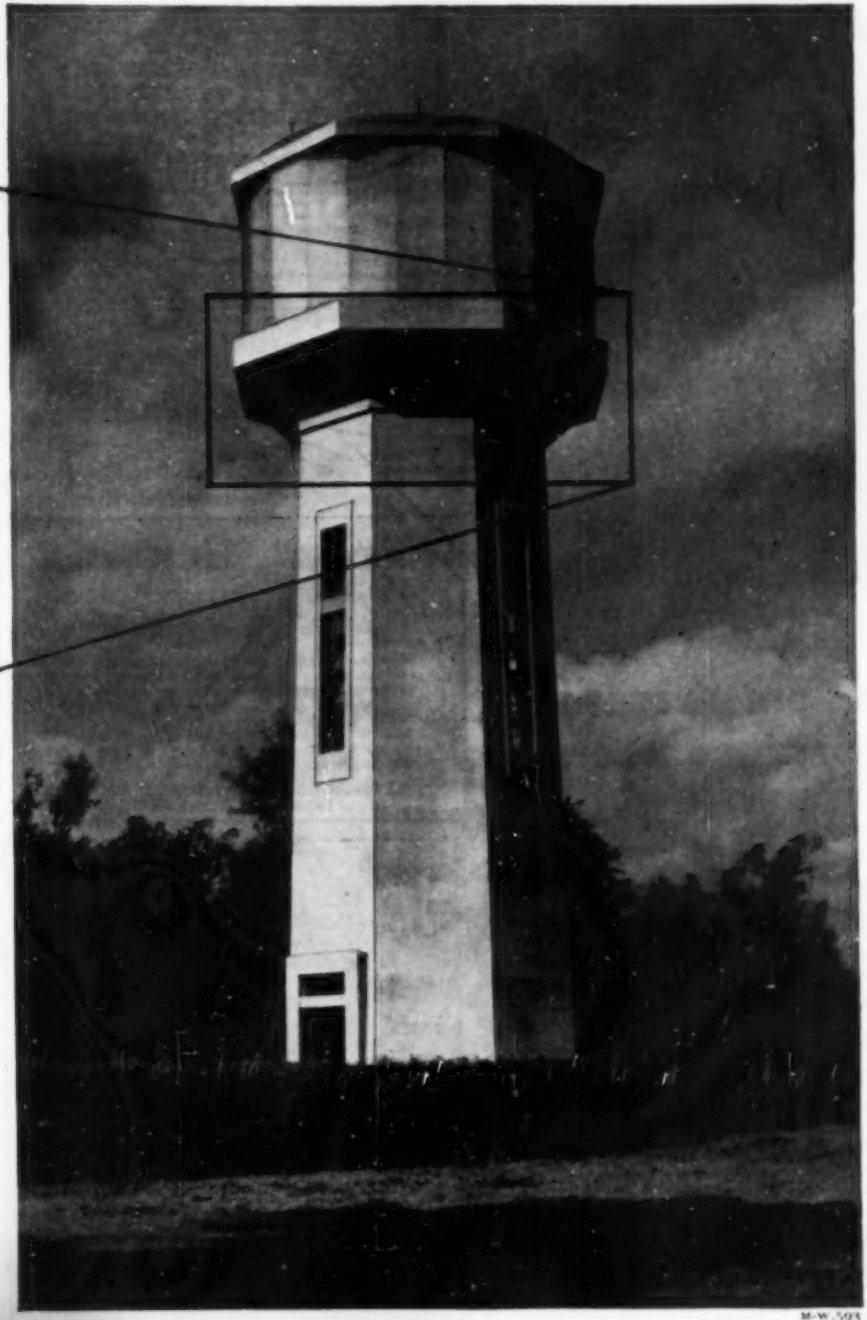

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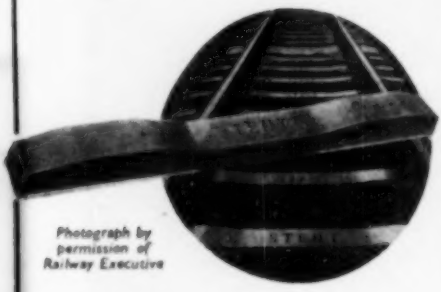

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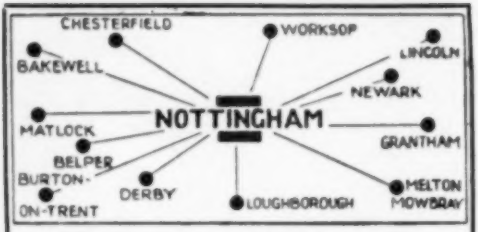

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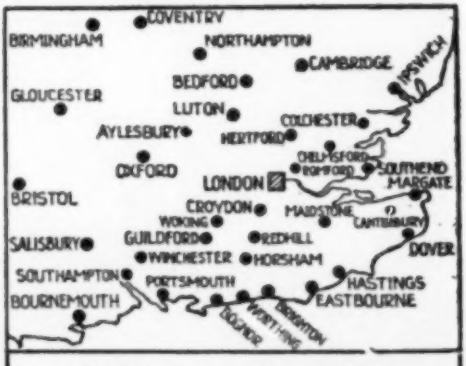

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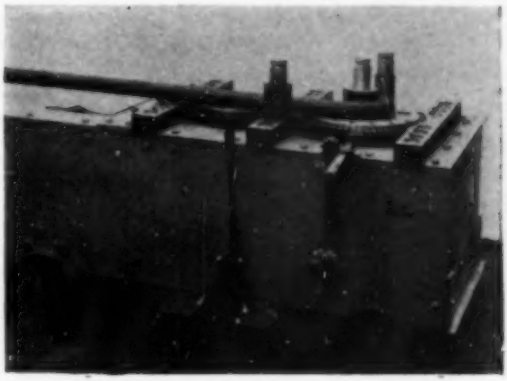

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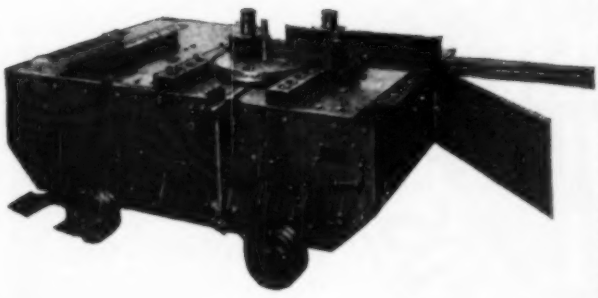

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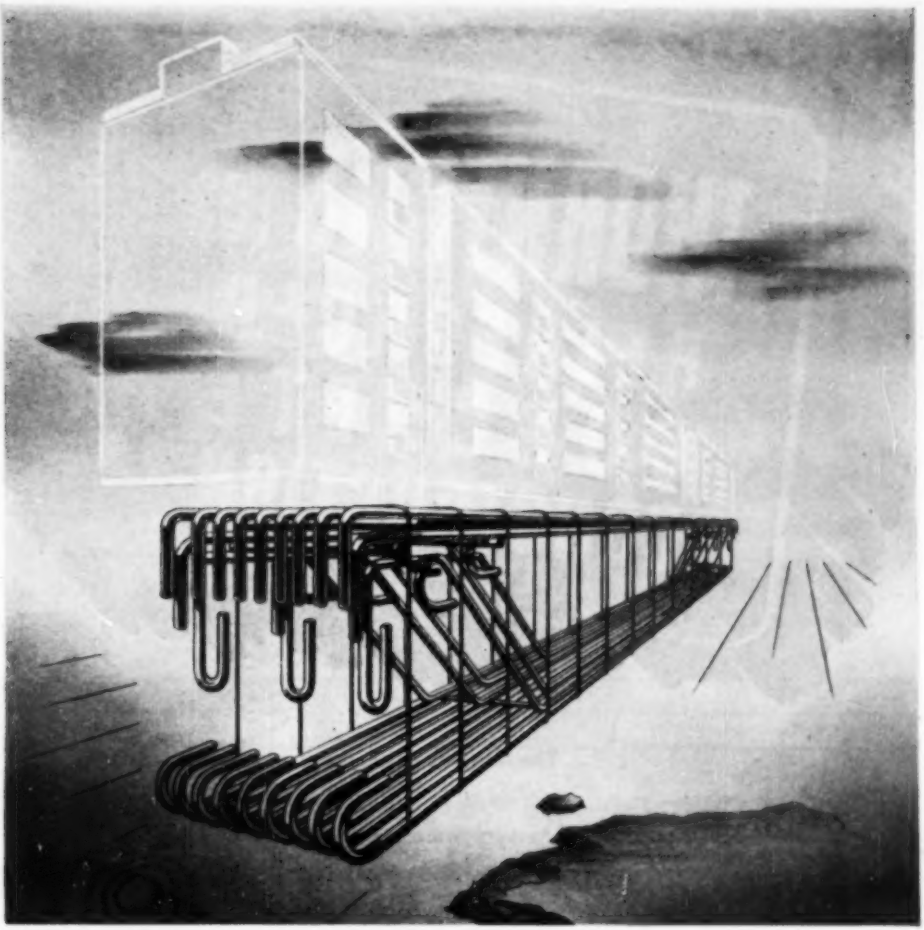

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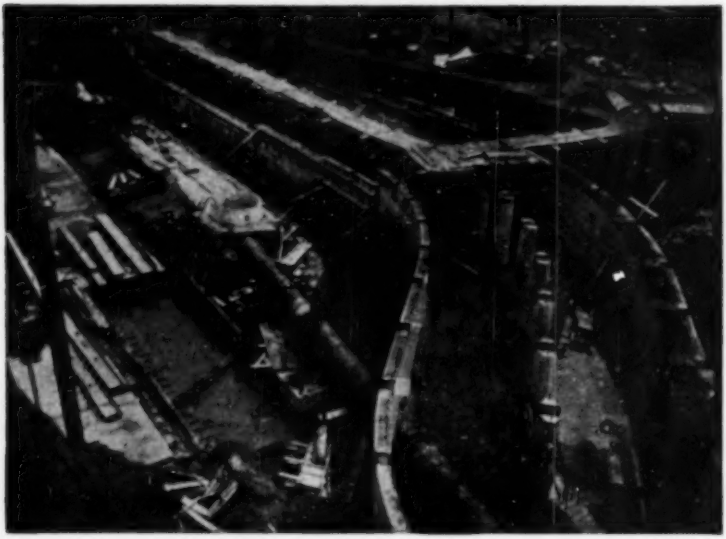

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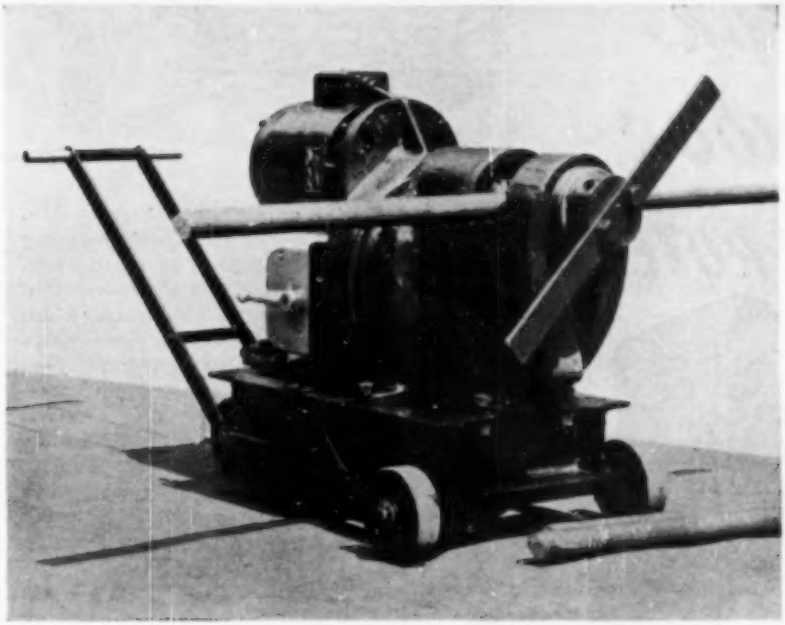

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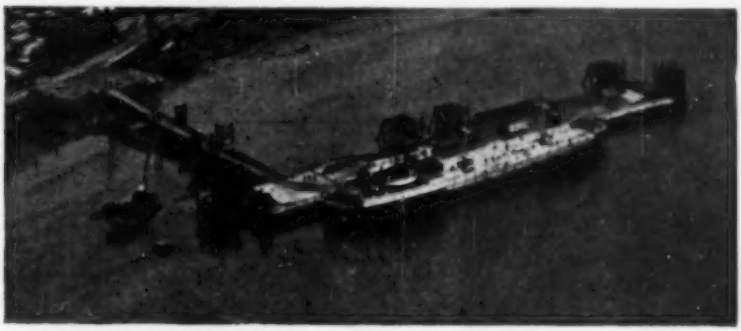

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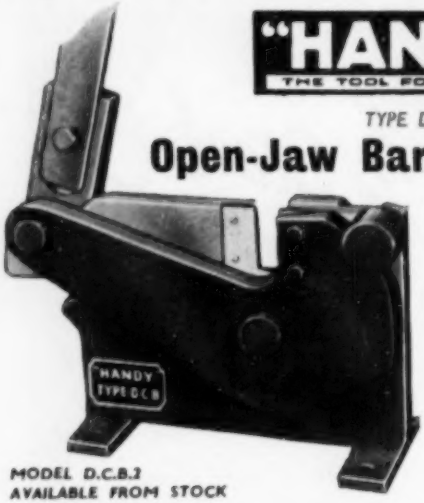

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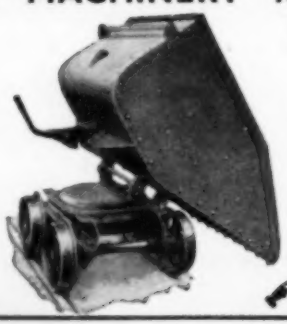

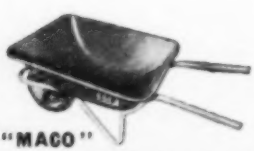

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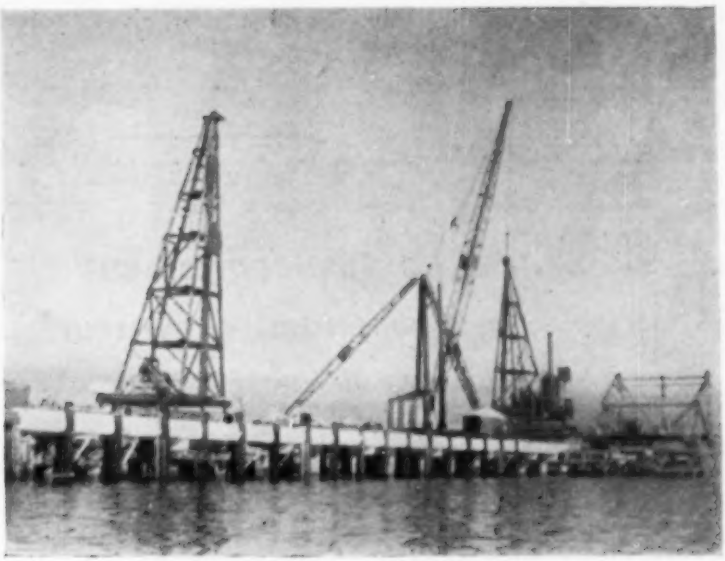

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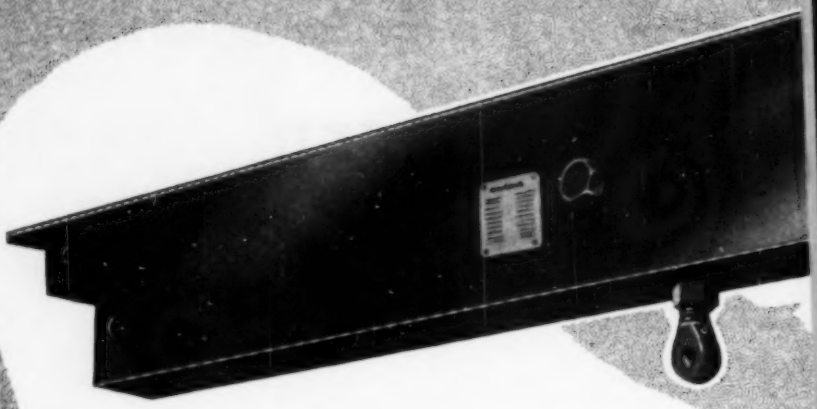

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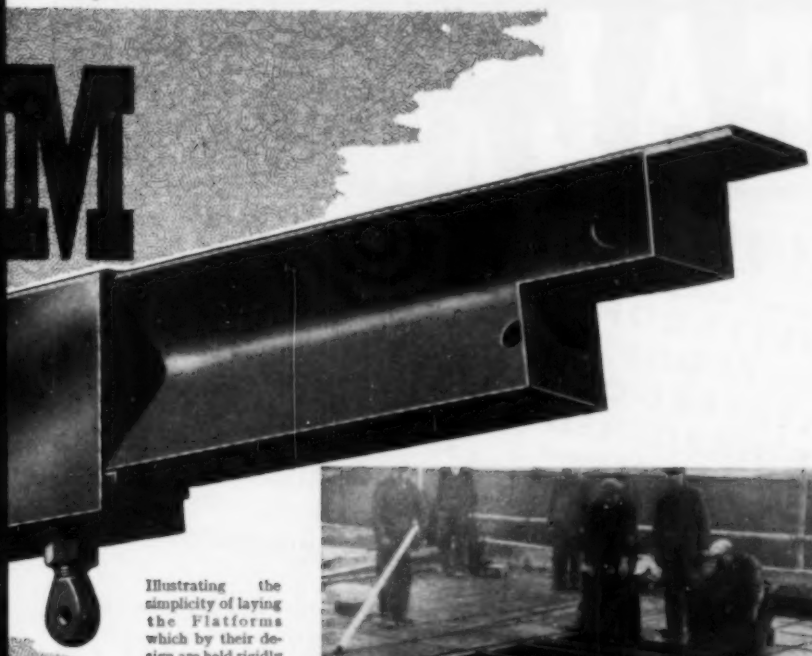

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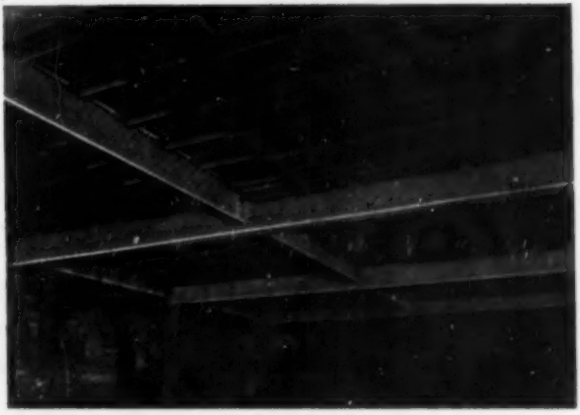

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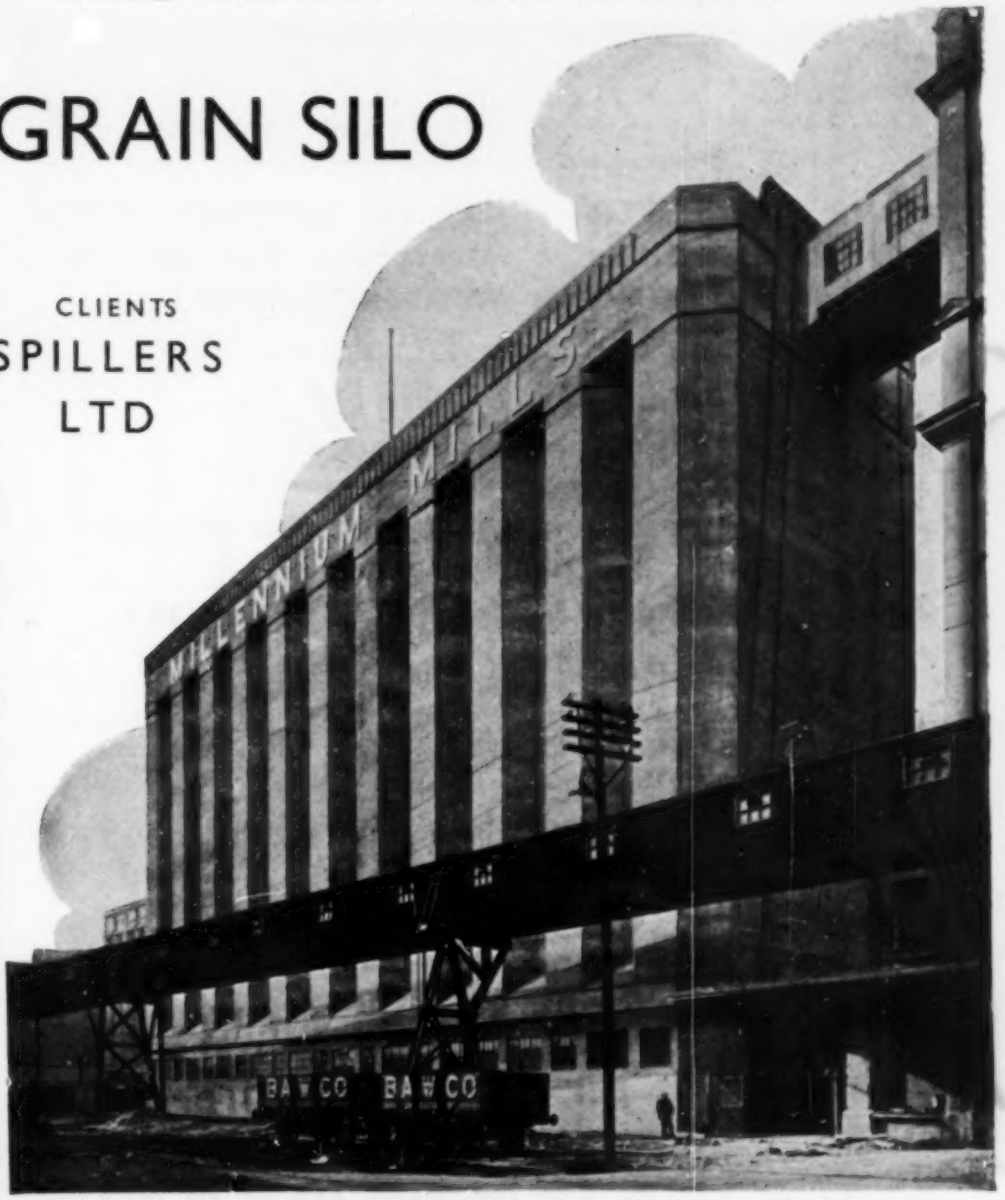

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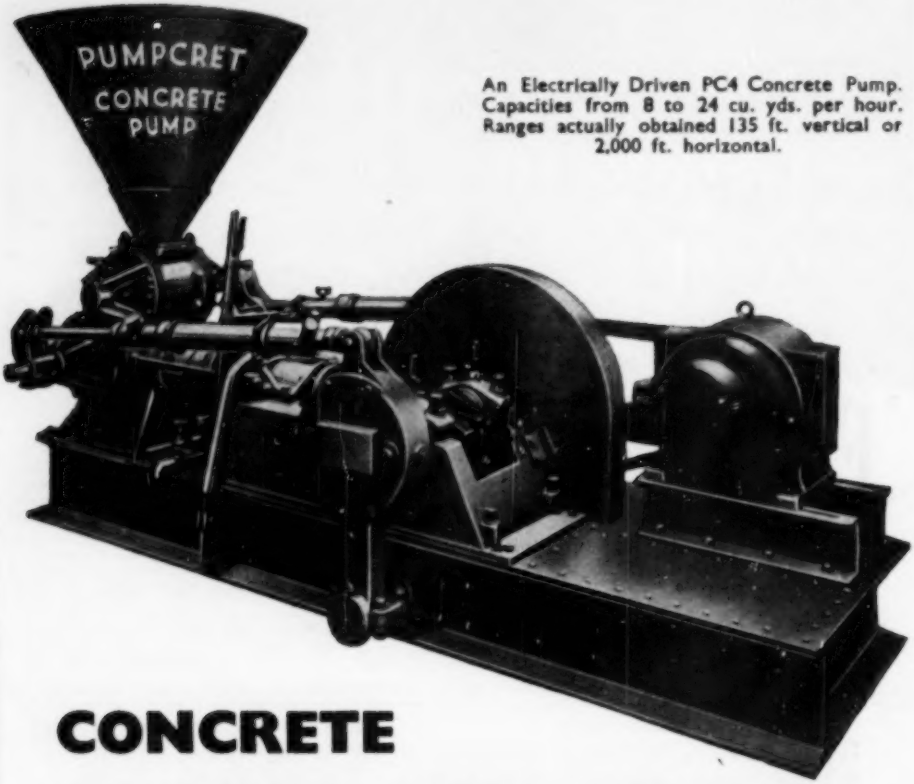

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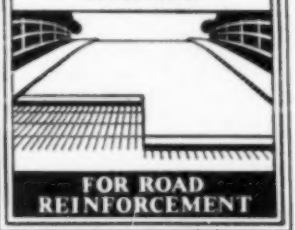

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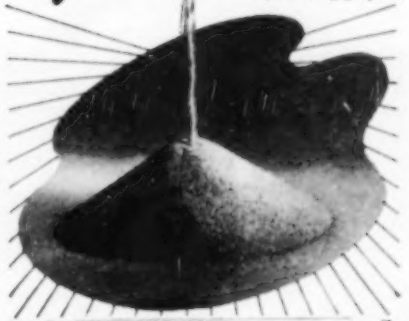

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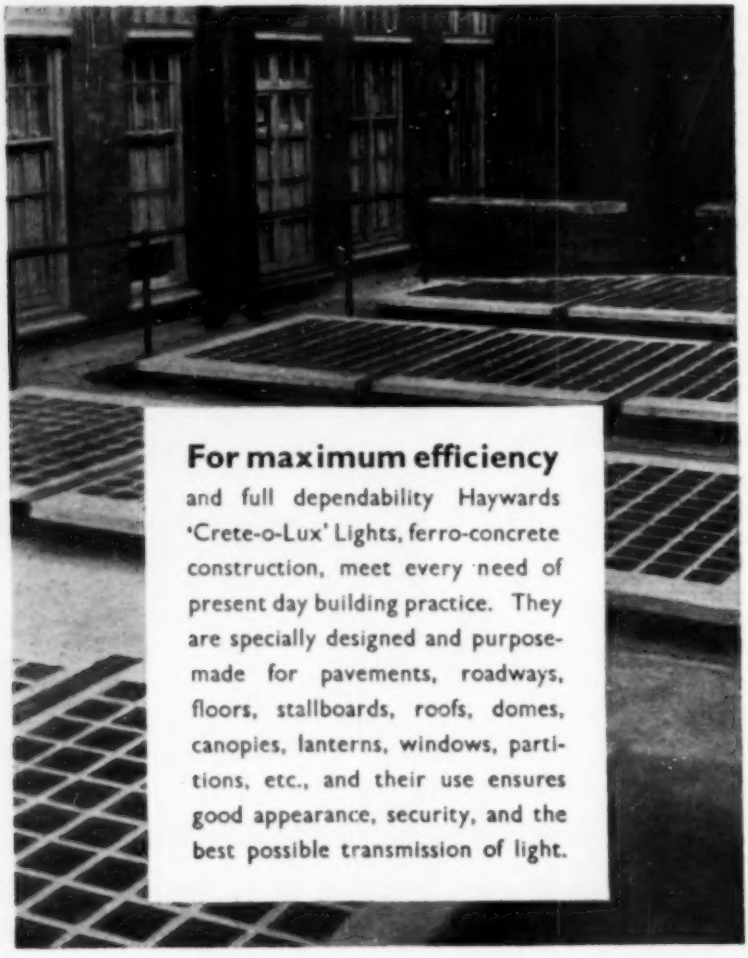

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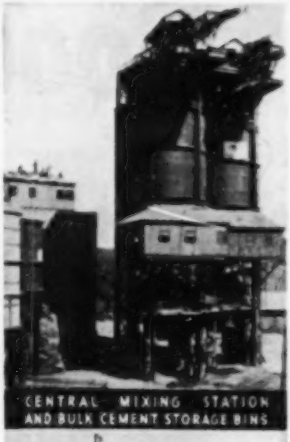

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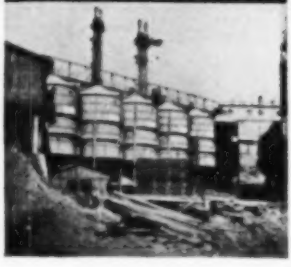

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CONCRETE
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Volume XLV. No. 2. LONDON, FEBRUARY, 1950

EDITORIAL NOTES

Interpretation of Test Results.

WE have often mentioned in these notes that in spite of codes of practice, regulations, and standard specifications, the solution of even ordinary problems in structural and civil engineering requires the exercise of judgment. This is particularly true when interpreting the results of strength tests of materials. No one would be satisfied with the result of only one test on a material it is intended to incorporate in a structure. Two tests may give a fairly true indication if the results are not widely different, but commonly three tests are made so that variations in quality or operational errors are more likely to be detected. The British Standard for mild steel bars for reinforcement requires only one tensile test to be made for each cast of bars up to ten tons (and a second test only if this weight is exceeded), but it is usual for two or more tests to be made. Although one test may be sufficient for a material made under strictly controlled conditions, for cement mortar and concrete, the specimens of which are made either on the site or in a laboratory of three or more constituents, three or more tests are essential, and the question then arising is how the results of the tests shall be considered so as to give reliable information.

The British Standard for Portland cement bases the tensile strength on the average strength of six briquettes. Compliance is doubtless obtained if some of the briquettes have a very low strength and some a very high strength, and unless the strength of each briquette is known it is impossible to decide whether the material is satisfactory or not. Large variations may be due, among other causes, to differences in the composition of the sample, or to variable quality of the material being tested. The latter of these causes is the most important as one of the main purposes of the tests is to determine whether or not the material is sound. Large variations are ignored in the American method of interpreting tensile-test results by neglecting the strengths of all faulty briquettes and of those having strengths 15 per cent. more or less than the average strength of all the briquettes tested. The average strength of the remaining specimens (of which there must be at least three) is then considered to represent the tensile strength of the mortar. A similar process of selection is adopted in considering the results of compressive tests of mortar cubes, but the limiting variation from the average strength of all cubes tested is 10 per cent. By the British standard method, the average of the compressive strengths (expressed to the nearest multiple of 25 lb. per square inch) of three mortar cubes represents the strength.

The examination of the strength of each specimen in a batch of concrete cubes is even more important. The British Standard Code (and some other codes and regulations) define the compressive strength of concrete as the average of

three cubes provided that the difference between the greatest and least does not exceed 15 per cent. of the average of the strengths of the three cubes. Consider ordinary structural concrete site-made cubes which must have a compressive strength of not less than 3000 lb. per square inch. Two extreme cases, the results of which are acceptable in accordance with the Code, are : (i) Two cubes of 3150 lb. per square inch and one of 2700 lb. per square inch. (ii) Two cubes of 2850 lb. per square inch and one of 3300 lb. per square inch. The concrete in (i) would doubtless be acceptable, the low strength of one cube being attributed possibly to a defective cube, but, assuming the cubes to be properly made, the concrete in (ii) should not be accepted as a material having a strength of not less than 3000 lb. per square inch.

The foregoing is based mainly on the consideration of three results of which two are nearly the same and one is different, that is the acceptance of the closest two out of three. Recently, however, research at the United States National Bureau of Standards seems to show that in some cases this rule is fallacious and leads to less precise results than the averaging of three results. If two of three results are about the same and the third differs considerably, it is common to suspect, and perhaps reject, the third. The question the Bureau is endeavouring to answer is, how different may the third result be before it should be suspected ? When two results are the same or very nearly so, the ratio of the differences between adjacent results may be very high, and it is then generally clear that the third result is due to an operational accident. Such a case is that of three cubes (the results of which are acceptable for 3000-lb. concrete) having strengths of 2835 lb., 2885 lb., and 3280 lb. per square inch, for which the ratio is about 8. Other examples are the foregoing cases (i) and (ii) for which the ratio is infinity. If the ratio is unity (as in 2775 lb., 3000 lb., and 3225 lb.) all the results must obviously be included, that is the " two-out-of-three " rule cannot be applied. With a ratio of 5 : 1 (for example, 2825 lb., 2900 lb., and 3375 lb.), the border line between the average of three results or two results seems to be approached for a 3000-lb. concrete. Study at the Bureau of sets of three results known not to contain gross errors shows that a ratio of 9 or more can occur in 20 per cent. of the cases considered. The report of this study does not say to what material or type of tests the results considered applied. With a variable material such as concrete a suitable border-line ratio can be assessed only by considering the difference between the lowest and highest result expressed as a percentage of the average (the smaller the percentage difference the greater the border-line ratio), and the average strength compared with the specified strength. The more the average strength exceeds the specified strength the greater may be the acceptable ratio, and in the not uncommon case of the lowest strength being greater than the specified strength, variations in the results are generally ignored. When some of the strengths, however, are below the specified strength of the concrete, consideration of the results of tests on the foregoing lines of accepting some and discarding other results is important if the probable strength is to be ascertained. It may even be that if the strength of one specimen out of three is below that specified, it should be assumed that the concrete does not meet the requirements, because the vagaries of selecting and testing specimens will generally be less than the vagaries of workmanship on the site, and if one-third of the concrete tested has less than the strength required it is highly probable that the strength of much of the concrete in the work will also be less than the strength required.

The Design of Reinforced Concrete Pressure-Vessels.

By S. W. LEWAREN.
(CHICAGO.)

CYLINDRICAL vessels used in chemical plants are frequently made of concrete, they are generally subjected to high temperatures, and the internal pressure may be greater or less than the atmospheric pressure Owing to the proportions of such a pressure-vessel, and particularly because the wall and ends are commonly thick compared with the diameter, the body must be designed as a thick cylinder, the effect of the restraint the ends impose on the free deformation of the wall should not be ignored, and the stresses due to temperature must be taken into account. As an example the design of the vessel shown in *Fig.* 1*a* is given in the following.

Fig. 1*b* shows in an exaggerated manner the deformations due to an internal pressure. The lateral expansion of the cylindrical body of the vessel is due to the circumferential or hoop tension in the wall. The vertical expansion is due to the internal pressure on the ends causing longitudinal extension of the wall. The internal pressure on the ends also causes bending moments on the circular slabs forming the ends, and the restraint at the junction of each end and the wall causes a bending moment of opposite sign to occur around the periphery of the ends and in the wall. *Fig.* 1*c* shows (also exaggerated) the deformations due to an external pressure which may be due to the pressure in the vessel being less than atmospheric pressure.

Analysis as a Thick Cylinder.

A vessel having the dimensions shown in *Fig.* 1*a* is assumed to be subjected to an internal pressure p of 75 lb. per square inch and the uniform temperature inside the vessel is assumed to be 200 deg. F.

It is necessary first to determine a provisional thickness of, and the reinforcement in, the wall of the vessel so as to provide sufficient resistance to cracking, that is so that the tensile stress in the concrete does not exceed, say, 175 lb. per square inch as recommended in the "Code of Practice for Reinforced Concrete Structures for the Storage of Liquids" published by the Institution of Civil Engineers. The circumferential tension is about $12pR_2 = 12 \times 75 \times 48 = 43{,}200$ lb. per foot of depth. The thickness of the wall should be about 43,200 lb. $\div$ (12 $\times$ 175 lb. per square inch) = 20·6 in.; because of the reduction in ring tension due to restraint by the end slabs and the reduction in the stress in the concrete due to the reinforcement the wall can be less thick, say 18 in. For sufficient resistance to ring tension, should the concrete crack, the amount of reinforcement in a ring 1 ft. deep must be not less than 43,200 lb. $\div$ 12,000 lb. per square inch = 3·6 sq. in., which can be provided in three bands (*Fig.* 2). Due to temperature stresses (as described later) the outer band must contain more reinforcement than the middle and inner bands, and the reinforcement can be provisionally $\frac{7}{8}$-in. bars at 3-in. centres (2·41 sq. in.) in the outer band, and $\frac{3}{4}$-in. bars at 3-in. centres (1·23 sq. in.) in the middle and inner bands, giving a total of 4·87 sq. in. per foot of depth.

The wall must now be analysed as a thick cylinder. From Lamé's theory, the circumferential tensile stress in an elementary ring of radius x (*Fig.* 2) in a homogeneous wall of a thick cylinder is

$$c_{tx} = \frac{pR_2^2}{R_1^2 - R_2^2}\left(\frac{R_1^2}{x^2} + 1\right),$$

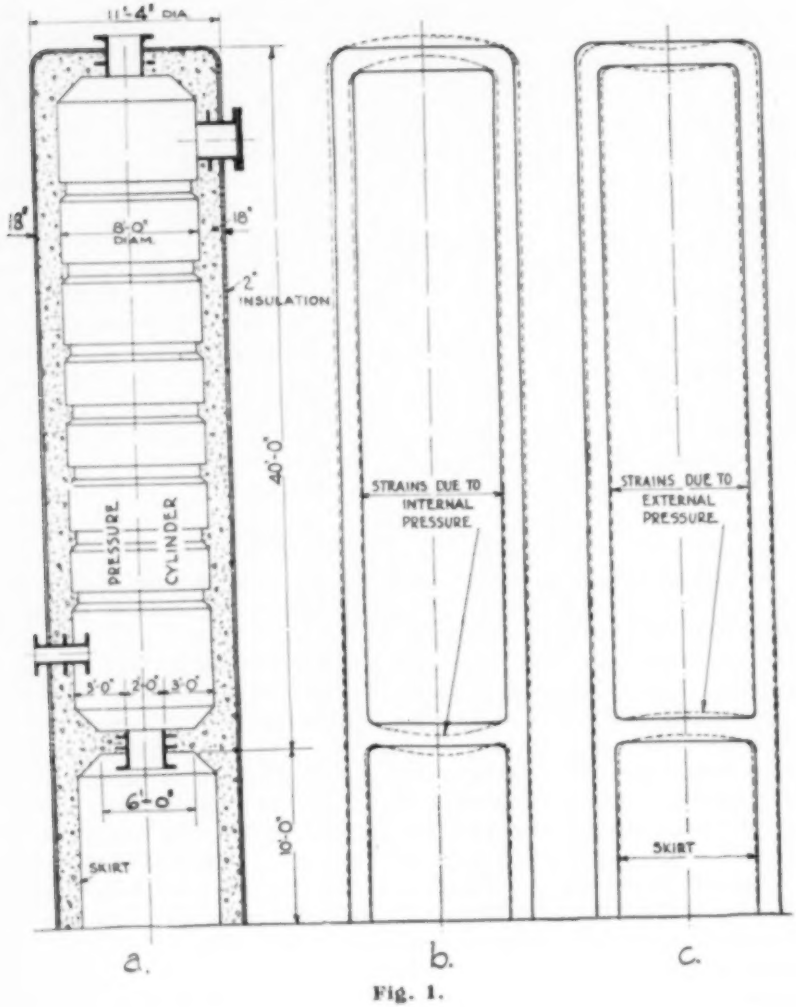


Fig. 1.

the internal pressure p and the stress c_{tx} being in the same units. Substituting numerical values in pounds and inches for the vessel under consideration, $c_{tx} = 84{\cdot}5\left(\frac{66}{x^2} + 1\right)$ lb. per square inch. Neglecting the reinforcement, the

magnitude and distribution of the theoretical tensile stresses across the thickness of the wall are given in *Fig.* 2. The stress varies from a maximum of 243 lb. per square inch at the inner face to a minimum of 169 lb. per square inch at the outer face. The effect of the reinforcement in each band can be allowed for approximately by multiplying the stresses by $\frac{12d'}{12d' + A_T'(m-1)}$, where A_T' is

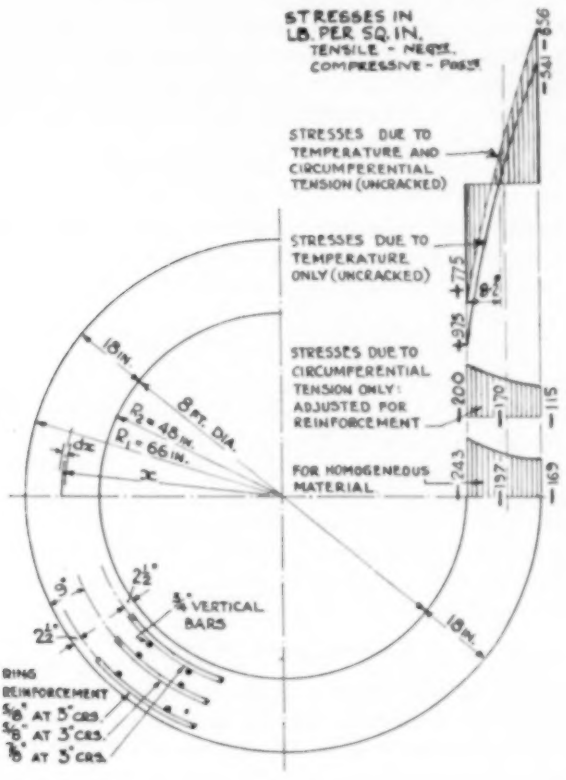


Fig. 2.

the area of the reinforcement in one of the three bands in 1 ft. depth of wall and d' is the thickness of the strip of wall assumed to be reinforced by one band. Therefore with a modular ratio $m = 12$ (elastic modulus for 1 : 1½ : 3 concrete in tension being, say, $2{\cdot}5 \times 10^6$ lb. per square inch), the reduced tensile stress at the inner face ($d' = 5$ in.) is about $0{\cdot}82 \times 243 =$ say, 200 lb. per square inch; at the middle of the wall ($d' = 8$ in.), $0{\cdot}88 \times 197 =$ say, 170 lb. per square inch; and at the outer face ($d' = 5$ in.), $0{\cdot}70 \times 164 = 115$ lb. per square inch. These theoretical stresses, which are those likely to occur should the internal pressure operate without the high temperature, are reasonably close to the maximum accepted tensile stress of 175 lb. per square inch required to minimise the risk of cracking the concrete.

It is necessary to ascertain that, should the concrete crack, the tensile stress in the circumferential reinforcement is not excessive. The total circumferential tension in the concrete in a ring of the wall 1 ft. deep is

$$12\int_{R_3}^{R_1} c_{tx}\,dx = 12 \times 84{\cdot}5\int_{48}^{66}\left(\frac{66^2}{x^2}+1\right)dx = 43{,}400 \text{ lb.},$$

which is about the same as the approximate value of $12pR_2$. The average tensile stress in the reinforcement replacing the cracked concrete is 43,400 lb. ÷ 4·87 sq. in. = 8,900 lb. per square inch, but the greatest stresses occur in the inner and middle bands. Assuming that the inner band is in the centre of a strip 5 in. wide, the circumferential tension to be resisted, when the concrete is cracked, is

$$1014\int_{48}^{53}\left(\frac{66^2}{x^2}+1\right)dx = 14{,}200 \text{ lb.},$$

and the stress in the reinforcement is 14,200 ÷ 1·23 = 11,500 lb. per square inch. Assuming that the middle band resists the circumferential tension in a strip 8 in. wide in the middle of the wall, the tension to be resisted is

$$1014\int_{53}^{61}\left(\frac{66^2}{x^2}+1\right)dx = 19{,}300 \text{ lb.},$$

and the stress is 19,300 ÷ 1·23 = 15,700 lb. per square inch. These maximum stresses, which may occur should the concrete crack and the temperature be normal, are not excessive, especially as the stress in the bars nearer the inner surface is within the limiting stress of 12,000 lb. per square inch generally permitted in cylindrical liquid-containing structures in this condition, and the restraint by the ends of the cylinder is ignored. Therefore at normal temperature the design is satisfactory.

Temperature Stresses.

When heat is applied uniformly along the entire inner surface of a thick cylinder, thereby causing a difference of temperature between the inner and outer faces of the wall, the ring shape remains constant, but a compressive stress is induced at the inner face and a tensile stress at the outer face, a bending moment being created across the wall. In a homogeneous material the stress, which acts circumferentially, when analysed by applied mechanics,* is

$$c_{Tx} = \frac{E_c T\varepsilon}{2(1-\mu)\log_e \frac{R_1}{R_2}}\left[1 - \log_e \frac{R_1}{x} - \frac{R_2^2\left(1+\frac{R_1^2}{x^2}\right)}{R_1^2 - R_2^2}\log_e \frac{R_1}{R_2}\right],$$

where T is the difference in temperature (deg. F.) between the inner and outer faces, ε is the coefficient of linear thermal expansion (0·000006 per deg. F.), μ is Poisson's ratio (say, 0·15 for concrete), and E_c is the elastic modulus (say, $3{\cdot}5 \times 10^6$ lb. per square inch for $1:1\frac{2}{3}:3\frac{1}{3}$ concrete in compression, and

* J. N. Goodier: "Thermal Stress." Journal of Applied Mechanics, Vol. IV, No. 1, March, 1937.

$2{\cdot}5 \times 16^6$ lb. per square inch in tension). The difference in temperature is calculated from

$$T = \frac{(T_i - T_0)d}{k_c\left(\frac{d}{k_c} + \frac{h}{k_h} + a_0\right)},$$

where T_i is the temperature of the contents of the vessel (assumed to be 200 deg. F.), T_0 is the air temperature (minimum, say, 20 deg. F.), d is the thickness of the wall (18 in. in the example), k_c is the conductivity of concrete (7 B.Th.U. per square foot per hour per deg. F. per inch of thickness), h is the thickness of the insulation (say, 2 in.), k_h is the conductivity of the insulation (assumed to be 0·5), and a_0 is the coefficient of the outer surface (say, 0·2). By substitution, T is about 70 deg. F., and at the inner face ($x = 48$ in. and $E_c = 3{\cdot}5 \times 10^6$ lb. per square inch) the compressive stress is 975 lb. per square inch and at the outer face ($x = 66$ in., $E_c = 2{\cdot}5 \times 10^6$ lb. per square inch) the tensile stress is 541 lb. per square inch, assuming that the concrete has not cracked. By equating $\left[1 - \log_e \frac{R_1}{x} - \frac{R_2^2\left(1 + \frac{R_1^2}{x^2}\right)}{R_1^2 - R_2^2} \log_e \frac{R_1}{R_2}\right]$ to zero, it is seen that, for the numerical values in this example, $c_{Tx} = 0$ when $x = 56{\cdot}2$ in. These stresses are shown in *Fig.* 2 and, when these are combined with the tensile stresses due to the circumferential tension, the maximum tensile stress is 656 lb. per square inch, which is likely to cause the concrete to crack, but as the cracks would be in the outer face (there being a compressive stress of 775 lb. per square inch at the inner face), the tightness of the vessel would not be impaired. When it is cracked the resistance of the vessel is dependent in the first instance on the outer band of reinforcement, and it is necessary to show that in this condition this reinforcement is not too highly stressed. This can be done approximately in the following manner.

The strains corresponding to the stresses of + 775 lb. and − 656 lb. per square inch are those that would be produced on an uncracked rectangular isotropic section 18 in. deep and 12 in. wide by a central tensile pull of 10,000 lb. and a bending moment of 400,000 in.-lb. The stresses in the cracked section can be considered as those due to the same force and bending moment, and the tensile stress t in the outer band of reinforcement can be calculated approximately thus :

$$e = \frac{400{,}000}{10{,}000} = 40 \text{ in.}; \quad e_s = 40 + 9 - 2{\cdot}5 = 46{\cdot}5 \text{ in.}$$

$$t = \frac{10{,}000}{4{\cdot}87} + \frac{10{,}000 \times 46{\cdot}5}{0{\cdot}875 \times 15{\cdot}5 \times 2{\cdot}41} = 2060 + 14{,}250 = 16{,}310 \text{ lb. per square inch,}$$

which is not excessive in reinforcement near the outer face of the wall.

These calculations neglect the restraining effect of the ends and the stresses due to the shrinking of the concrete. Shrinking causes additional tensile stress in the concrete and a reduction of the tensile stress in the reinforcement. The effect of the ends is to restrict the outward deformation of the cylinder and

therefore to reduce the tensile stresses in the wall. Thus the neglected effects are both favourable to the reinforcement and, as regards the tension in the concrete, their influences are contrary and therefore largely offset each other.

Design of the Ends.

The ends of the cylindrical vessel are each analysed as a circular disk with a hole at the centre and restrained around the edges. For the bottom end, which is an annular ring 3 ft. wide (effective width a, say, 24 in. because of the 15-in. splays) and of thickness $h = 16$ in., the maximum compressive stress due to bending * is

$$\frac{1{\cdot}09Kpa^2}{h^2} = 1{\cdot}09 \times 2{\cdot}23 \times 75 \times 24^2 \div 16^2 = 407 \text{ lb. per square inch}$$

due to the uniform pressure, and for a load distributed along the periphery of the central opening (which is the condition when the cover of the manhole is closed) the maximum stress is

$$\frac{1{\cdot}09Kp}{h^2} = 1{\cdot}09 \times 0{\cdot}993 \times 33{,}800 \div 16^2 = 144 \text{ lb. per square inch.}$$

For the thermal stress in the end slab the bending moment per inch of the circumference is, for a restrained edge, $\frac{E_c \varepsilon T(1+\mu)h^2}{12(1-\mu^2)}$. Assuming that the temperature of the underside of the bottom slab within the enclosing skirt is 150 deg. F., $T = \frac{(200-150)16}{7\left(\frac{16}{7}+0{\cdot}2\right)} = 46$ deg. F., and by substitution the moment is 17,500 in.-lb. per inch (with $E_c = 2{\cdot}5 \times 10^6$) and the maximum compressive stress due thereto is $1{\cdot}09 \times 6 \times 17{,}500 \div 16^2 = 448$ lb. per square inch. There is, however, a direct tension, due to the restraint between the wall and the bottom, of 75 × (say) 48 in. ÷ 2 = 1800 lb. per inch of circumference, inducing an average tensile stress of $\frac{1800}{16} = 112$ lb. per square inch. Therefore the total maximum compressive stress is 407 + 144 + 448 − 112 = 887 lb. per square inch. The calculation of this stress assumes that the slab is homogeneous and not cracked. The effect of the reinforcement is to reduce slightly (as in the wall) the values of the component stresses, except that due to the difference in temperature, and it is reasonable to assume that the summation of the compressive stresses in the bottom of the slab at the splay is therefore 350 + 125 + 450 − 100 = 825 lb. per square inch. The corresponding tensile stress in an uncracked slab is 350 + 125 + 450 + 100 = 1025 lb. per square inch, which greatly exceeds the probable tensile strength of the concrete, which will therefore crack and the tensile resistance must be provided by reinforcement. These stresses in a 16-in. slab are equivalent to those produced by an axial tensile force of 19,200 lb. per foot of width and a bending moment of 473,000 in.-lb. per foot of width. There-

* See "Strength of Materials," by S. Timoshenko; Part II, page 156. The values $K = 2{\cdot}23$ and $0{\cdot}993$ are taken from this source and the factor 1·09 is introduced by the writer to allow for the non-homogeneity of reinforced concrete.

fore $e = \frac{473{,}000}{19{,}200} = 24{\cdot}6$ in., and $e_s = 24{\cdot}6 + 8 - 1{\cdot}5 =$ say, 31 in. The area of tensile reinforcement required is $\frac{19{,}200}{12{,}000}\left(1 + \frac{31}{0{\cdot}875 \times 14{\cdot}5}\right) = 5{\cdot}54$ sq. in. per foot, say, 1½-in. bars spaced radially at 4 in. around the periphery of the slab. (A tensile stress of 12,000 lb. per square inch should not be exceeded in these bars, as the cracks occur on the inner face of the slab.) Compressive reinforcement is not essential, but will be provided by the extension of the tensile reinforcement at the middle at the bottom of the slab, say, ⅝-in. bars at 6-in. centres in two directions.

The shearing force per inch at the edge of the splay is $\frac{75\pi \times 33^2}{2\pi \times 33} = 1240$ lb. and the intensity of shearing stress is $1240 \div (0{\cdot}875 \times 14) = 100$ lb. per square inch, which does not exceed the permissible shearing stress for 1 : 1⅔ : 3⅓ concrete.

Vertical Reinforcement.

The vertical reinforcement in the wall resists the direct tension due to the internal pressure on the ends which is $75 \times 3{\cdot}14 \times 48^2 = 543{,}000$ lb. The area of reinforcement to be distributed uniformly around each of the three bands of horizontal steel in the wall is $\frac{543{,}000}{3 \times 12{,}000} = 15$ sq. in., that is, say, 36 ¾-in. vertical bars to each band. For very tall and exposed vessels additional vertical reinforcement may be needed to resist wind stresses, but in most structures the weight of the concrete is sufficient to retain the walls in compression.

External Pressures.

For a vessel subjected to external pressure due to a vacuum or otherwise, the analysis is similar to the foregoing if the formulæ and the sense of the stresses take account of the inwardly acting pressure. For practical purposes it is sufficient to use the simple expression to compute the compressive stress in the wall, but it is advisable to check the maximum compressive stress that occurs at the inner surface from the expression for a thick cylinder, $\frac{2pR_1^2}{R_1^2 - R_2^2}$. If the higher temperature is inside the vessel this compressive stress will be increased and the compressive stress in the outer face decreased by the temperature stresses. The stresses in the end slabs will be the reverse (except for temperature) of those for a vessel with internal pressure, and tensile stresses can be considerably reduced by dishing the ends and designing them as spherical, elliptical, or conoidal concrete domes with an opening at the crown.

Construction.

To make the vessels gas-tight and liquid-tight, a dense 1 : 1⅔ : 3⅓ concrete is advisable and the water-cement ratio should not exceed 0·5. With thick walls as in the example described, the coarse aggregate can be a maximum size of 2½ in. Crushed fire-brick is a suitable coarse aggregate for structures subjected to heat. The concrete in the entire vessel should be cast continuously from the

skirt upwards, without construction joints. It is recommended that the concrete should be placed at the rate of about 4 ft. vertically per hour and not more than 5 ft. per hour, and to avoid separation it should not be dropped more than 2 ft. 6 in. into the shuttering. The greatest period between successive lifts should not exceed 30 minutes. Curing should begin as soon as possible after casting. The entire inner and outer faces of the vessel must be maintained moist for at least three days, or longer in warm weather. The faces can be watered or have saturated hessian hung against them. An hydraulic head should not be permitted to build up against the inner faces, as the green concrete may be too weak to resist the pressure.

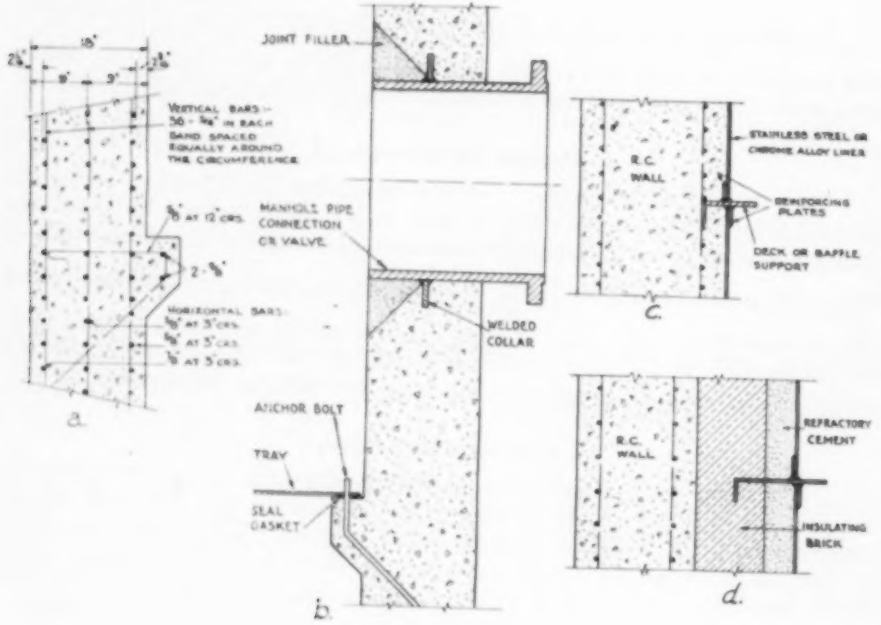


Fig. 3.

Upon completion of curing, joints at the manholes, valves, and other fittings which are put in position before the concrete is placed must be made airtight by pointing with a joint filler (*Fig. 3b*), a suitable rebate having been formed in the face of the wall for this purpose. The filler should be a mastic of the consistency of grout and should contain a densifying agent, such as fly-ash, and an expansive admixture such as one containing iron-filings. The wall of a concrete vessel containing corrosive vapours at moderate temperature and pressure can be protected by a metal lining as shown in *Fig. 3c*. If the temperature is very high an insulating lining is necessary as in *Fig. 3d*. In neither of these designs in *Figs. 3 c* and *d* is shuttering required for the inner face of the wall.

Concrete Construction at Abbey Steel Works, South Wales

THE new Abbey Works of the Steel Company of Wales at Margam, near Port Talbot, occupies a site 2¼ miles long by ¾ mile wide. The principal building is ¾ mile long, and in it are the soaking-pit furnaces, slabbing mill, reheating furnaces, continuous mill, hot-strip mill, and finishing departments. The two other main groups of buildings are the melting shop and the cold-reduction mill. All construction at and below ground level is of concrete, most of which is reinforced. The superstructures of the main buildings are of structural steel with corrugated steel-sheet sides and insulated aluminium roofs. The roofs are flat with alternate high and low bays, each low bay having panels of glazing vertically from roof to ground. *Figs.* 1 to 5 are views of the foundation work and ancillary structures.

The site, which is separated from the sea by sand dunes, was low-lying marshy land drained by ditches leading to a pumping station discharging into tidal water. The level of the land has been raised about 10 ft. and this work required about 4,000,000 cu. yd. of sand, gravel, slag, and other filling materials, some of which were placed dry and others were deposited hydraulically. On the seaward side a protective littoral dune is being raised about 20 ft. above the foreshore;

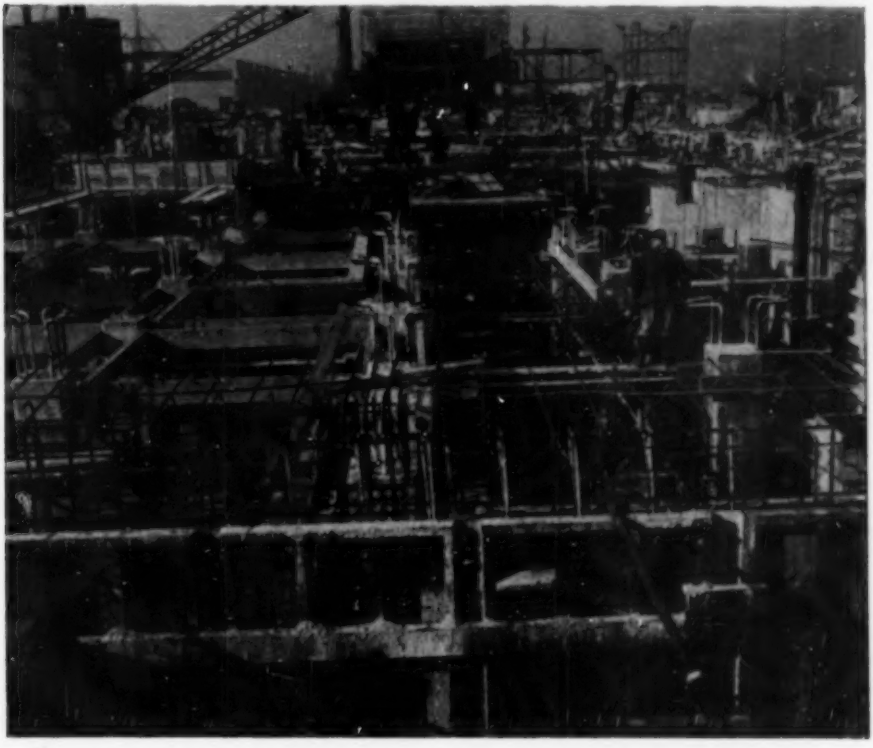


Fig. 1.—Construction of Floor over Basement of Slabbing Mill.

the sand is being stabilised by planting special grasses, and later a belt of trees will be grown. Below the natural bed of sand and clay lie strata of peat and ballast overlying rock. The sides of deep excavations were retained with steel sheet-piles some of which are left in place owing to the difficulty of extracting them. The foundations are generally on 7 ft. 6 in. deep. The shafts of the stanchions are embedded in concrete, for which purpose pockets with corrugated sides were formed in the bases. A base in which the stanchion has been set in position, but not yet concreted, is shown in *Fig. 2*, which also shows the brickwork used as shuttering for some of the work. The provision of brick linings permitted

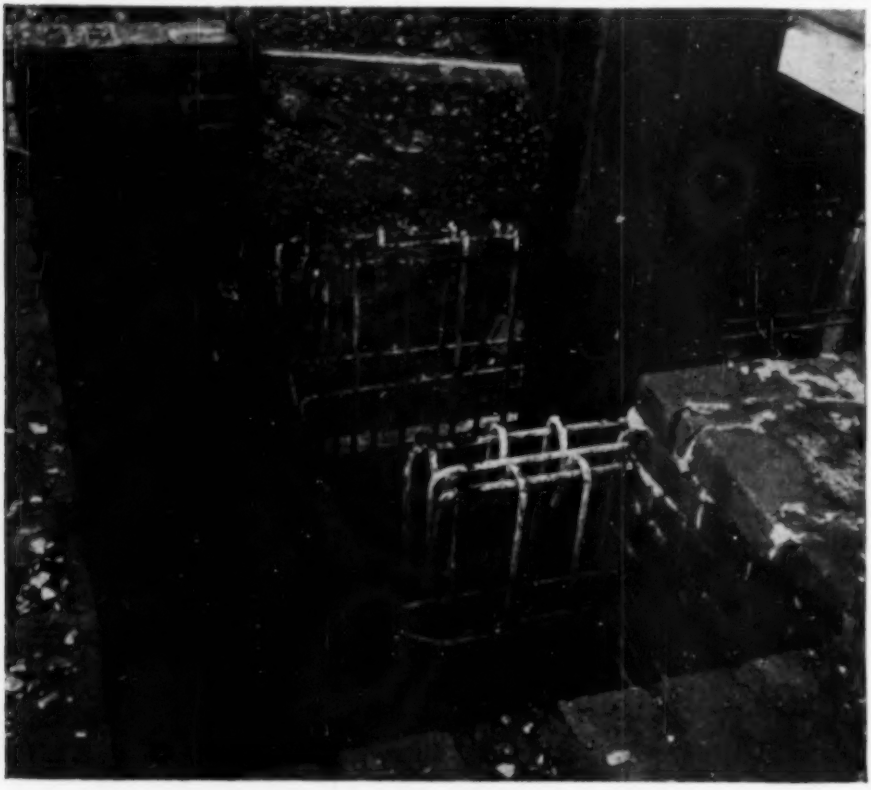


Fig. 2.—Foundation for Stanchion.

cast-in-situ piles, and more than 30,000 piles, 20 ft. to 45 ft. long, have been formed and about 3000 have yet to be placed. The larger piles carry a load of 90 tons. More than 120 piles have been subjected to load tests. Some of the largest column bases support 4200 tons as well as resisting bending moments from stanchions. The lower part of these bases is 39 ft. 6 in. long, 22 ft. wide, and cavities to be prepared well in advance of filling without employing joiners, and retained the ground in excavations.

Not much timber shuttering was used, steel plates with steel-channel walings and stiffeners being principally used for vertical faces. For horizontal faces steel pans and adjustable props and soffit supports were used.

Many hundreds of bolts, some up to

6 in. diameter, have to be set in the concrete and the dimensions from the centre-lines of each group must be generally within $\frac{1}{16}$ in. of those specified; the dimensions from the centre of each group to the main setting-out lines must also be within $\frac{1}{8}$ in. The method adopted for holding the bolts in position until the concrete has hardened is shown in *Fig. 2*, and comprises two steel channels placed back to back and supported on a steel frame fitted to the top of the preceding lift of concrete. The bolt is placed between the channels and is suspended from a template through which it passes. The clearance between the channels permits small adjustments of the position of the bolt to be made by lateral movement.

The reinforced concrete floors are mostly designed for a superimposed load of 2000 lb. per square foot, but in parts loads of from 400 lb. to 4000 lb. per square foot occur. Where suspended floors to carry the greatest loads are in panels 10 ft. square, the slabs are 15 in. thick and are reinforced with $\frac{7}{8}$-in. bars at 10-in. centres. Where the slabs span 25 ft. they are haunched at the supports and are 2 ft. thick at the centre, and are reinforced with $1\frac{1}{4}$-in. bars at 6-in. centres.

Concrete.

In the foundations and other concrete construction at ground level there are about 340,000 cu. yd. of concrete. Ordinary Portland cement is delivered in bags for the concrete that is batched by volume, and in bulk in wagons for concrete batched by weight. The aggregate is medium hard carboniferous limestone from the Steel Company's quarry at Cornelly, seven miles from the site and is in three gradings, namely, $1\frac{1}{2}$ in. to $\frac{3}{16}$ in., $\frac{3}{8}$ in. to $\frac{1}{16}$ in., and $\frac{1}{16}$ in. to dust. The fine sand from the dunes adjacent to the site is also used, and the grading of this material is such that 98 per cent. passes a No. 52 B.S. sieve (0·012 in.) and 10 per cent. passes a No. 100 B.S. sieve (0·006 in.). Other fine aggregate is grit dredged from the Bristol Channel and graded from $\frac{1}{16}$ in. to No. 52 B.S. sieve.

Batching by volume has been used for most of the concrete, and about two-thirds of this concrete has been mixed in continuous mixers. The principal mixture

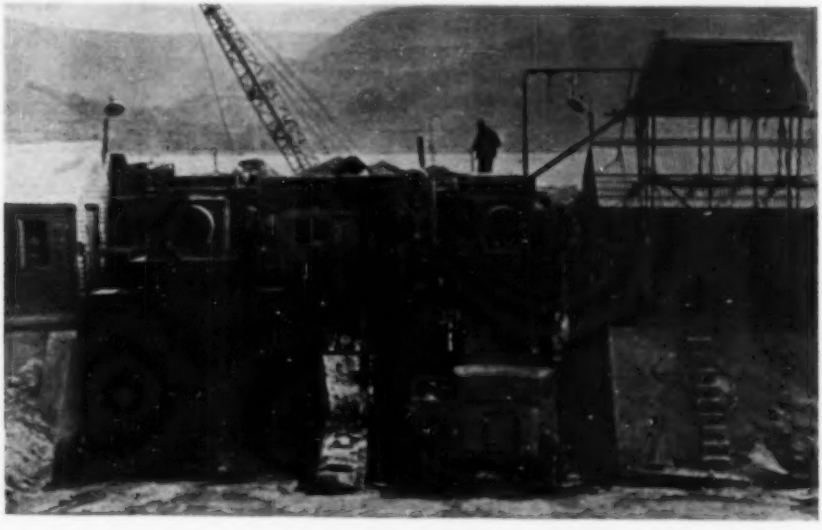


Fig. 3.—Continuous Mixers.

used is 1 part (by volume) of cement to 6 parts of combined aggregate, the coarse aggregates being either 1½-in. to ⅛-in. or ¾-in. to ⅛-in. limestone and the fine aggregate being a mixture of grit and dune sand. There have been three installations of continuous mixers for most of the work, one of which comprised two mixers (*Fig.* 3), and the other two installations had one mixer each. Materials are loaded by dragline into hoppers at the sides of the continuous mixers and the materials are measured by worm-conveyors driven at speeds which are adjusted to suit each mixture. Materials from the worm-conveyors are fed to bucket-elevators discharging continually into the mixers.

For the cold-reduction mill, a weigh-batching plant with two 1-cu.-yd. rotating-drum mixers is used, and this will mix about 50,000 cu. yd. of concrete. The mixture is generally 1 : 6½ by weight with ¾-in. to ⅛-in. or 1½-in. to ⅛-in. coarse aggregate. The fine aggregate is a mixture of dune sand and limestone ⅛ in. to dust, blended while being discharged from hoppers with vibrators attached.

The concrete is distributed by pumps through 6-in. and 4-in. pipelines. At first the pumps were supplied directly from the mixers, but now the pumps are generally mobile and the concrete is delivered to them by lorries. A considerable amount of concrete has been placed from bottom-opening buckets of 1 cu. yd. and 2 cu. yd. capacity handled by mobile cranes. Special high-lift lorries (*Fig.* 4) with hopper-shaped bodies are used for transporting concrete to the pumps and to outlying parts of the site. The greatest amount of concrete placed in one week is 3500 cu. yd.

The average strength of cubes of ordinary concrete is 3600 lb. per square inch. There is a laboratory on the site for testing materials, checking concrete mixtures, crushing cubes, and other routine tests. Experiments are also carried out in connection with problems arising on the site. For example tests were made to show the resistance to attack by acid of various mixtures of concrete. Investigations of refractory concrete resulted in a mixture of 1 : 6 (by volume) of crushed firebrick being used for the 6-in. to 9-in. protective layer of concrete over the furnace foundations. Investigations have also been made on the problem of grouting successfully the bases of heavy machines.

Fig. 4.—Hopper-trucks for Transporting Concrete.

Fig. 5.—Water-Treatment Plant.

Although many parts of the work are subjected to great and sudden changes of temperature, permanent expansion and contraction joints are not provided even in the soaking pits, which are about 480 ft. long. Shrinkage gaps 1 ft. to 3 ft. wide were formed as concreting proceeded, and where possible were concreted at least three weeks after the main concrete had been placed. The gaps were left at about 40-ft. centres or at changes in cross section where cracking might be expected. In some parts slabs have been concreted by alternate-bay construction.

Ancillary Works.

Works ancillary to main buildings include concrete roads, railways, ducts, drains, bridges, etc., and structures in connection with the water supply, including a reservoir, pump-house, water tower, and cooling tower (*Fig.* 5). The reservoir is in a depression formed by the removal of dune sand by pumping, the sand being carried through pipes for use mainly as hydraulic filling elsewhere on the site. The area of water in the reservoir will be eventually about 18 acres, and the average depth about 12 ft. The pump-house is 44 ft. wide and 34 ft. deep, and is mainly below ground level. The concrete walls are 2 ft. 6 in. thick and are strutted near the top by horizontal reinforced concrete members which act as struts and also as beams carrying the floor.

The cooling tower is a hyperbolic reinforced concrete structure 265 ft. high, and is 160 ft. in diameter at the base and 92 ft. at the neck. The thickness of the wall at the top and middle is 5 in., and 21 in. at the bottom. The cooling capacity is 1,800,000 gallons per hour.

The consulting engineers for the structural and civil engineering work are Messrs. W. S. Atkins & Partners. The main contractors for the foundations of the main building and the melting shop

are Messrs. George Wimpey & Co., Ltd., and for the cold-reduction mill Messrs. John Laing & Son, Ltd. The contractors for most of the piling are the Franki Compressed Pile Co., Ltd.

The cooling tower was designed by the Davenport Engineering Co., Ltd., and erected by Messrs. Holst & Co., Ltd.

At Trostre, about 25 miles west of Margam, works for the manufacture of tin-plate from the strip made at Abbey Works are in course of construction. Piled foundations and concrete work at and below ground level are extensive and similar in some respects to those at Margam.

Continuous Mixing of Concrete.

At a meeting of the Reinforced Concrete Association held last month, Mr. W. G. Furlonger gave some particulars of the continuous mixers in use on the site of the Abbey Works and which are shown in *Fig.* 3.

Comparison of the results of tests of compressive strengths of a large number of cubes of volume-batched concrete and those of continuously-mixed concrete shows that continuous mixing can give results at least as accurate as those of volume-batched mixing in machines of the same capacity operated under the same supervision. The coefficient of variation at Abbey Works with continuous mixers was 18 per cent., which is about the same as the results from volume batching.

The continuous mixers, which are portable, have at the rear separate hoppers for fine and coarse aggregate. Through the bottom of each hopper run two screw-feeds passing through control gates and feeding the material into a boot. A small hopper for cement also has a screw-feed discharging into the boot, from which the dry and partly mixed materials are lifted by a bucket-elevator and discharged into the mixing drum. The drum has five rows of blades each of which is about 7 ft. long and 2 ft. 8 in. in diameter. The drum revolves continuously, concrete being discharged continuously at the rate of about 25 cu. yd. per hour. Water is sprayed continuously into the drum at the inlet end and is controlled by a hand-operated valve. The water is in a tank on the top of the machine.

The screws feeding the material into the hopper are fitted with driving sprockets of different sizes and revolve at a rate proportional to the size of the sprocket which controls therefore the proportions of the materials in the mixture. Variations in the shape, moisture content, and grading of the aggregate can cause considerable variation in flow. If the aggregate is very fine mechanical or manual vibration may be needed to prevent arching of the material in the hopper. During a period of six months and under similar conditions, the greatest variation in the flow of coarse aggregate was $\pm$ 10 per cent., but was generally 5 per cent. On the plant at this site, the grit is fed from a side hopper on to the screw conveying the sand, and the greatest variation of flow of sand and grit was plus 10 per cent. and minus 8 per cent. Generally the range was $\pm$ 5 per cent. The variation in flow of cement was $\pm$ 5 per cent., but generally the range was 2 per cent.

The amount of cement is constant if the amount in the hopper is kept reasonably constant, if the cement is obtained from one source, and the variation of temperature is not great. Cement may often be hot when delivered and tests show that up to a temperature of 100 deg. F. the flow should be constant but above this temperature variation took place and hot cement is not used until it has cooled. The flow of rapid-hardening Portland cement seems to bear no relation to that of ordinary Portland cement, and the machine had to be reset for each type of cement. Analyses during a period of a year gave an average mixture of 1 to 5·9 compared with the required mixture of 1 to 6. Generally there was a larger variation above the mean strength than below it.

The machine is set by loading separately stone and sand into a calibrated lorry or hopper and taking the time required to put through a measured amount. The quantity of material entering the boot is controlled by two penstocks, the initial setting of which is done by trial and error.

Design of Reinforced Concrete Members in accordance with the British Standard Code.—II*.

By CHAS. E. REYNOLDS, B.Sc., A.M.Inst.C.E.

Solid Slabs.

SOLID slabs can be designed in the same manner as rectangular beams with b equal to 12 in. if the bending moment M is in inch-pounds per foot width of slab. The effective depth required is given by $k_2\sqrt{M}$, and values of k_2 for the commonly-permitted stresses are given in *Table* No. 4. If the effective depth provided is less than that calculated, the slab should be designed in the same way as a rectangular beam ($b = 12$ in.) in the same circumstances.

If the effective depth provided is equal to or is not much greater than that calculated, the design procedure is as follows:

With mild steel reinforcement and c not exceeding 1000 lb. per square inch, the tensile value, $T_V(= A_T \times 0{\cdot}85 \times 18{,}000)$ lb. per foot width per inch of effective depth, must not be less than $\frac{M}{d}$. If c is not to exceed 1100 lb. per square inch, the tensile resistance, $T(= A_T \times 18{,}000)$ lb. per foot width, must not be less than $\frac{M}{0{\cdot}84d}$. Values of T_V and T are given in *Table* No. 4 for bars of common sizes at various spacings.

With high-tensile reinforcement in which t may be 27,000 lb. per square inch, and if c is not to exceed 1000 lb. per square inch, the lever arm is $0{\cdot}88d$ and the tensile value, $T'_{V1}(= A_T \times 0{\cdot}88 \times 27{,}000)$ lb. per foot width per inch of effective depth, must not exceed $\frac{M}{d}$. If c is not to exceed 1100 lb. per square inch, the tensile resistance, $T'(= A_T \times 18{,}000)$ lb. per foot width, must not be less than $\frac{M}{0{\cdot}84d}$. If plain round high-tensile bars are used, *Table* No. 4 can be used by selecting reinforcement having T_{V1} (at 18,000 lb.) not less than $\frac{2}{3}T'_{V1}$, or having T (at 18,000 lb.) not less than $\frac{2}{3}T'$, as the case may be.

A slab can also be designed for the maximum stresses of $t = 18{,}000$ lb. and $c = 1000$ lb. per square inch by using *Table* No. 3, which gives the moments of resistance and the areas of reinforcement of slabs of different thicknesses reinforced in tension only. If the actual combination of cover of concrete and diameter of bar results in an effective depth differing from those tabulated, the data for the slab having an effective depth equal to or just less than the actual effective depth should be applied. It is rarely economical to space bars in slabs at the greatest permissible pitch of $3d$ if the stress in the concrete is near the greatest permitted. The larger bars, and consequently the greater cover of concrete, cause a considerable reduction in the effective depth. A reasonable design is produced if the size of the bar is such that the spacing is from 4 in. to 8 in. *Table* No. 3 has been compiled on this basis.

Generally the thickness of a slab is much greater than the minimum thickness required and, as for rectangular beams, in this case the Code permits the lever

* Continued from January, 1950. *Tables Nos.* 1 to 4 are given in the previous article.

arm to be assumed to be $0{\cdot}88d$ so long as the amount of mild steel reinforcement does not exceed 1 per cent. and of high-tensile reinforcement not more than 0·67 per cent. These limiting percentages occur if $Q\left(=\frac{M}{d^2}\right)$ does not exceed $12 \times 155 = 1860$ with mild steel and $12 \times 158 = 1896$ with high-tensile steel. The tensile value of mild steel reinforcement, $T_{V1}(= A_T \times 0{\cdot}88 \times 18{,}000)$ lb. per foot width per inch of effective depth, must be not less than $\frac{M}{d}$. The tensile value of high-tensile reinforcement, $T'_{V1}(= A_T \times 0{\cdot}88 \times 27{,}000)$ lb. per foot width per inch of effective depth, must be not less than $\frac{M}{d}$; if plain round bars of high-tensile steel are provided, *Table* No. 4 can be used to select suitable reinforcement having T_{V1} (at 18,000 lb.) not less than $\frac{2}{3}T'_{V1}$.

If the Concrete is Less Strong than Standard Concrete.

If the strength of 1 : 2 : 4 concrete is less than that permitting a working compressive stress of 1000 lb. per square inch in beams and slabs, the maximum permissible stress, subject to certain conditions, is 750 lb. per square inch. This stress, a tensile stress of 18,000 lb. per square inch, and a modular ratio of 15 are a combination that was frequently used at one time, and charts and other aids for calculation are generally available. The tensile resistances T (at 18,000 lb.) in *Tables* Nos. 1 and 4 are, however, applicable. The lever arm is $0{\cdot}87d$ and d must be not less than $0{\cdot}089\sqrt{\frac{M}{b}}$ for beams and $0{\cdot}0257\sqrt{M}$ for solid slabs.

Stresses due to Wind.

When the stresses due to wind are included, the Code accepts the common practice of permitting the maximum stresses in the concrete and reinforcement to exceed by 33⅓ per cent. those ordinarily specified so long as the excess is solely due to the wind. One method is to design the beam or slab for the bending moment M excluding the wind, and then to calculate the stresses for the bending moment M_W which is M plus the effects of the wind. If the stresses due to M_W do not exceed by more than 33⅓ per cent. the stresses ordinarily permitted, the design is satisfactory. If otherwise, the member should be designed to resist M_W with the increased stresses, and it is then assured that the stresses due to M are less than the ordinary permissible stresses. A convenient method of designing a member to resist M_W at the higher stresses is to design the member to resist a bending moment of $0{\cdot}75M_W$ at the ordinary permitted stresses, for which purpose *Tables* Nos. 1 to 4 can be used as already described.

Examples of the Design of Beams and Slabs.

Methods of using *Tables* Nos. 1 to 4 are described in the examples that follow. The concrete in each case is 1 : 2 : 4 and the reinforcement mild steel. Other conditions are as stated in the sub-headings. Although certain numbers of bars of certain sizes are given, it will be realized that these give only one solution and other factors, such as the requirements for resistance to shearing force, may make other numbers and sizes of equal resistance more convenient.

EXAMPLE No. 1.—Design a rectangular beam to resist a bending moment of 640,000 in.-lb., the maximum permissible stresses being (*c*) 1000 lb. and (*t*) 18,000 lb. per square inch.

(i) Maximum stresses occurring simultaneously; tensile reinforcement only.

(*a*) Assume b to be 9 in. From *Table* No. 1 the effective depth d must be not less than $0{\cdot}072\sqrt{\frac{640{,}000}{9}} = 19{\cdot}2$ in. The overall depth would therefore be 21 in., and d is 19·5 in. The value of T_v required is $\frac{640{,}000}{19{\cdot}5} = 32{,}800$ lb., which is given by three 1-in. bars.

(*b*) Assume b to be 9 in. The moment of resistance required per inch of width is $\frac{640{,}000}{9} = 71{,}100$ in.-lb. From *Table* No. 3 the effective depth is between 19 in. and 20 in., say 19·5 in., the overall depth being therefore 21 in. The area of the reinforcement must be about $0{\cdot}245 \times 9 = 2{\cdot}21$ sq. in., which from *Table* No. 1 is given by three 1-in. bars.

(ii) Beam 24 in. deep and 12 in. wide.

$d = 22{\cdot}5$ in. $Q' = \frac{640{,}000}{12 \times 22{\cdot}5^2} = 105$, which is less than 155 so that the amount of mild steel reinforcement will be less than 1 per cent., and

$$T = \frac{640{,}000}{0{\cdot}88 \times 22{\cdot}5} = 32{,}300 \text{ lb.},$$

which from *Table* No. 1 is given by three $\frac{7}{8}$-in. bars.

(iii) Beam 18 in. deep and 11 in. wide.

$d = 16$ in. approximately. $Q' = \frac{640{,}000}{11 \times 16^2} = 227$, which is greater than 193. Alternative methods are:

(*a*) With reduced tensile stress.

From *Table* No. 2 ($c = 1000$ lb.) for $Q' = 227$, $p = 2{\cdot}35$ per cent.

$$A_T = 0{\cdot}01 \times 2{\cdot}35 \times 12 \times 16 = 4{\cdot}51 \text{ sq. in.},$$

say, four $1\frac{1}{4}$-in. bars (from *Table* No. 1).

(*b*) With compressive reinforcement.

Assume $a_s = 14$ in. $M_c = 193 \times 11 \times 16^2 = 543{,}000$ in.-lb.

$M_s = 640{,}000 - 543{,}000 = 97{,}000$ in.-lb.

$$c_s = 14{,}000\left(1 - \frac{2}{0{\cdot}455 \times 16}\right) = 10{,}150 \text{ lb. per square inch.}$$

$$A_c = \frac{97{,}000}{14 \times 10{,}150} = 0{\cdot}68 \text{ sq. in., say, two } \tfrac{3}{4}\text{-in. bars.}$$

$$T = \frac{543{,}000}{0{\cdot}85 \times 16} + \frac{97{,}000}{14} = 46{,}930 \text{ lb., which from } \textit{Table} \text{ No. 1 } (t = 18{,}000)$$

is given by two $1\frac{1}{8}$-in. and one $\frac{7}{8}$-in. bar. [It is seen that in this example there is about the same amount of reinforcement in designs (*a*) and (*b*) although it is arranged differently.]

(iv) The shallowest beam 10 in. wide.

For this case $A_T = A_c$. Moment of resistance per inch of width is

$$\frac{640{,}000}{10} = 64{,}000 \text{ in.-lb.}$$

From *Table* No. 3 ($A_T = A_c$), d is, say, 11½ in., and the overall depth is 13 in. By interpolation $A_T (= A_c)$ is, say, $0{\cdot}365 \times 10 = 3{\cdot}65$ sq. in.; say, three 1¼-in. bars in the top and bottom.

EXAMPLE No. 2.—Design a beam to resist a bending moment of 900,000 in.-lb. with maximum stresses not exceeding 27,000 lb. and 1100 lb. per square inch.

(i) Maximum permissible stresses occurring simultaneously.

Assume $b = 10$ in. From *Table* No. 1, d must not be less than

$$0{\cdot}074\sqrt{\frac{900{,}000}{10}} = 22{\cdot}2 \text{ in.};$$

total depth, say, 24 in.; $d = 22{\cdot}5$ in. T (at 27,000 lb.) $= \dfrac{900{,}000}{0{\cdot}87 \times 22{\cdot}5} = 46{,}000$ lb.; equivalent T (at 18,000 lb.) $= \frac{3}{5} \times 46{,}000 = 30{,}600$ lb., which is given by three ⅞-in. high-tensile round bars.

(ii) Beam 27 in. deep and 12 in. wide.

d is about 25·5 in. $Q' = \dfrac{900{,}000}{12 \times 25{\cdot}5^2} = 115$. Since this is less than 158 (*Table* No. 1), $T = \dfrac{900{,}000}{0{\cdot}88 \times 25{\cdot}5} = 40{,}200$ lb.; the equivalent at 18,000 lb. is $\frac{3}{5} \times 40{,}200 = 26{,}700$ lb., which from *Table* No. 1 is given by two 1-in. high-tensile bars.

EXAMPLE No. 3.—Calculate the reinforcement required in a tee-beam to resist a bending moment of 12,000 in.-lb. if the thickness of the slab is 6 in., the depth of the rib below the slab 18 in., and the width of the rib 8 in. The available breadth of slab is 56 in. The maximum stresses are 18,000 lb. and 1100 lb. per square inch ($n_1 = 0{\cdot}478$).

The effective depth is about $18 + 4 - 2 = 20$ in. $s_1 = \dfrac{4}{20} = 0{\cdot}2$. The depth to the neutral axis is $0{\cdot}478 \times 20 = 9{\cdot}6$ in.; therefore the neutral axis is below the flange. The lever arm a is $20 - 2 = 18$ in.

$T = \dfrac{1{,}200{,}000}{18} = 66{,}700$ lb. From *Table* No. 1, three 1¼-in. bars are suitable.

From *Table* No. 1, $x = 0{\cdot}695$ and $y = 1{\cdot}54$. The minimum breadth of flange b required is

$$\frac{1{,}200{,}000}{(1000 \times 20^2 \times 0{\cdot}2)[(0{\cdot}695 \times 0{\cdot}2^2) - (1{\cdot}54 \times 0{\cdot}2) + 1]} = 21 \text{ in.},$$

which is satisfactory.

EXAMPLE No. 4.—Design a slab to resist a bending moment of 45,000 in.-lb. per foot of width for the conditions stated.

(i) Maximum permissible stresses of 1000 lb. per square inch in the concrete and 18,000 lb. per square inch in the reinforcement induced simultaneously.

(*a*) From *Table* No. 3, the moment of resistance of a 5½-in. slab is 45,500 in.-lb. if $A_T = 0{\cdot}67$ sq. in.; say, ⅝-in. bars at 5½-in. centres (*Table* No. 4). Effective depth with ¾ in. of cover is 4·44 in. as in the table.

(*b*) From *Table* No. 4, $k_2 = 0{\cdot}0208$. Effective depth must be not less than $0{\cdot}0208\sqrt{45{,}000} = 4{\cdot}41$ in., that is a 5½-in. slab; $d = 4{\cdot}44$ in.

$$T_V = \frac{45{,}000}{4{\cdot}44} = 10{,}200 \text{ lb.};$$

say, ⅝-in. bars at 5½-in. centres (interpolated from *Table* No. 4).

(ii) Thickness of slab 6 in. ($d = 4{\cdot}94$ in.).

$$Q'' = \frac{45{,}000}{4{\cdot}94^2} = 1840;$$ as this is less than 1860, $a_1 = 0{\cdot}88$ and

$$T_{V1} = \frac{45{,}000}{4{\cdot}94} = 9{,}100 \text{ lb.},$$

which from *Table* No. 4 is given by ⅝-in. bars at 6½-in. centres.

The foregoing methods of designing solid slabs are also in accordance with British Standard Code of Practice CP. 114.101 (1950), "Suspended Concrete Floors and Roofs." According to this Code the cross-sectional area of transverse reinforcement (or distribution bars) in solid slabs must not be less than 0·1 of the cross-sectional area of the main reinforcement, a requirement that presumably supersedes, as regards the transverse reinforcement, the corresponding requirement in the Code for "Normal Reinforced Concrete" in which it is stated that the minimum amount of reinforcement in either direction must not be less than 0·1 per cent. of the cross-sectional area of the concrete.

(To be continued.)

The Late William Leslie Scott.

It is with deep regret that we announce the death of William Leslie Scott, which took place on January 23, in his sixtieth year, after a long illness. Scott was enthusiastic in his belief in the possibilities of reinforced concrete, and did much to further the use of this material during a period of forty years. He joined Considère Constructions, Ltd., in the year 1911. From 1914 to 1917 he was with the Public Works Department of the Gold Coast, and rejoined the firm in 1919. While Scott was chief engineer of Considère Constructions, Ltd., the firm was responsible for much important reinforced concrete work, including the Dorchester Hotel in London, the King George V bridge at Glasgow, and Chiswick and Twickenham bridges over the Thames. He was particularly interested in bridges, and published a book on reinforced concrete bridges in 1928. In the year 1936 Scott started in practice on his own account as a consulting engineer, and in 1945 took into partnership Mr. Guthlac Wilson, M.Inst.C.E., who is continuing the practice. Under the style Scott & Wilson the partnership was engaged on, among other important works, the new Concert Hall for the London County Council on the site of the Festival of Britain, a large factory in Wales for British Nylon Spinners, Ltd., a large housing estate at Pimlico for the London County Council, and roads and airfields in Nyasaland. Scott served on a number of committees dealing with concrete, notably the committee which drafted the D.S.I.R. Code of Practice for Reinforced Concrete in Buildings in connection with which, in conjunction with Dr. W. H. Glanville, C.B.E., he wrote an Explanatory Handbook which has become a standard work of reference.

Scott was an unassuming and most likeable man whose knowledge and experience can ill be spared.

Pioneers of Reinforced Concrete.

In connection with the celebration in France of the centenary of the invention of reinforced concrete, we referred in our number for December, 1949, to some of the pioneers in the development of reinforced concrete and their works. On the facing page are given portraits of some of these men. There seems to be no available portrait of J. L. Lambot, whose reinforced concrete boat illustrated on this page was built just before the year 1850 and is probably the first application of reinforced concrete. The houses shown on this page were probably built in 1852, and have roofs of concrete reinforced with iron bars of I section designed by

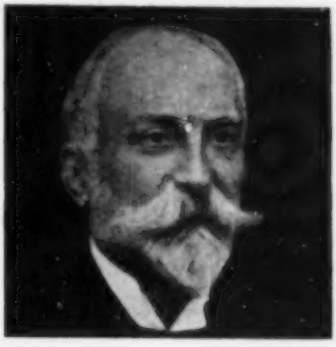
Reinforced Concrete Boat built before 1850.

Houses with Reinforced Concrete Roofs built in France about 1852.

François Coignet; this is the first known application of reinforced concrete in building construction. Monier took out a patent for reinforced concrete in the year 1867, and built a reinforced concrete water tank in the year 1872. Hennebique was a contractor rather than an inventor, and did much to develop the use of the material. Edmond Coignet read papers on the subject in 1889 and later, and to some extent specialised in reinforced concrete aqueducts and pipes. Mouchel designed the first reinforced concrete structures in Britain, built in the year 1897. Considère was chief engineer of the Roads Department of the Government of France before he specialised in reinforced concrete, and he was responsible for the formation of the commission which in 1906 produced the first French regulations for reinforced concrete.

As in the second half of the last century, so in the first half of this century, French engineers have been to the fore in inventing and developing new ideas in concrete design and construction, notably prestressed concrete.

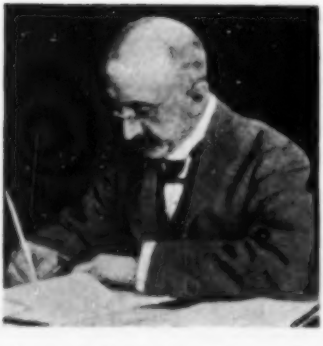
FRANÇOIS COIGNET.



EDMOND COIGNET.

JOSEPH MONIER.

FRANÇOIS HENNEBIQUE.



L. G. MOUCHEL.

ARMAND GABRIEL CONSIDÈRE.

Some Pioneers of Reinforced Concrete. *(See facing page.)*

A Reinforced Concrete Frame Power House.

A THIN-SLAB ROOF.

THE turbine house now being built at Skelton Grange power station, Leeds, is a reinforced concrete framed structure with a thin-slab vault roof stiffened by ribs. Half of the building (*Fig.* 1), measuring about 400 ft. by 79 ft., is being built now, and is divided by portal frames, of which the main roof ribs are part, into fourteen bays each 31 ft. long. The building will contain three 60,000-kw. turbo-alternators. In the middle of each bay a transverse secondary rib is provided below the roof slab. On one side the frames support a three-story reinforced concrete building containing the electrical equipment, and on the other the tanks which are at the level of the roof and which are also constructed in reinforced concrete. A large part of the turbine control floor, which is 30 ft. above ground, is also supported by the frames and is cantilevered from the main structure, but is isolated from the turbine blocks by a gap which prevents the transmission of vibration.

A 150-tons overhead travelling crane of 70-ft. 6-in. span traverses the length of the turbine house 30 ft. above the floor. The crane rails, the greatest load on which is 200 tons, are fixed to reinforced concrete beams supported by the frames. The columns of the frames are 5 ft. by 2 ft. 6 in. in cross section below the crane beams and 2 ft. 6 in. by 1 ft. 4 in. above.

Fig. 1.—Interior During Construction.

The point of contraflexure when the various loads act is at about the level of the crane beam. The bending moments at the springings of the ribs being small, the ribs are deeper at the middle of the span than at the springings. This condition is uncommon in a frame of this type, but has the advantage that the end clearance for the crane is increased. For the sake of appearance, the radius of the soffit of the ribs is the same as the radius of the roof slab. The columns project above the longitudinal gutter beams to stiffen the brick parapets and to provide resistance to wind pressure on the parapets. Large horizontal forces due to wind on the

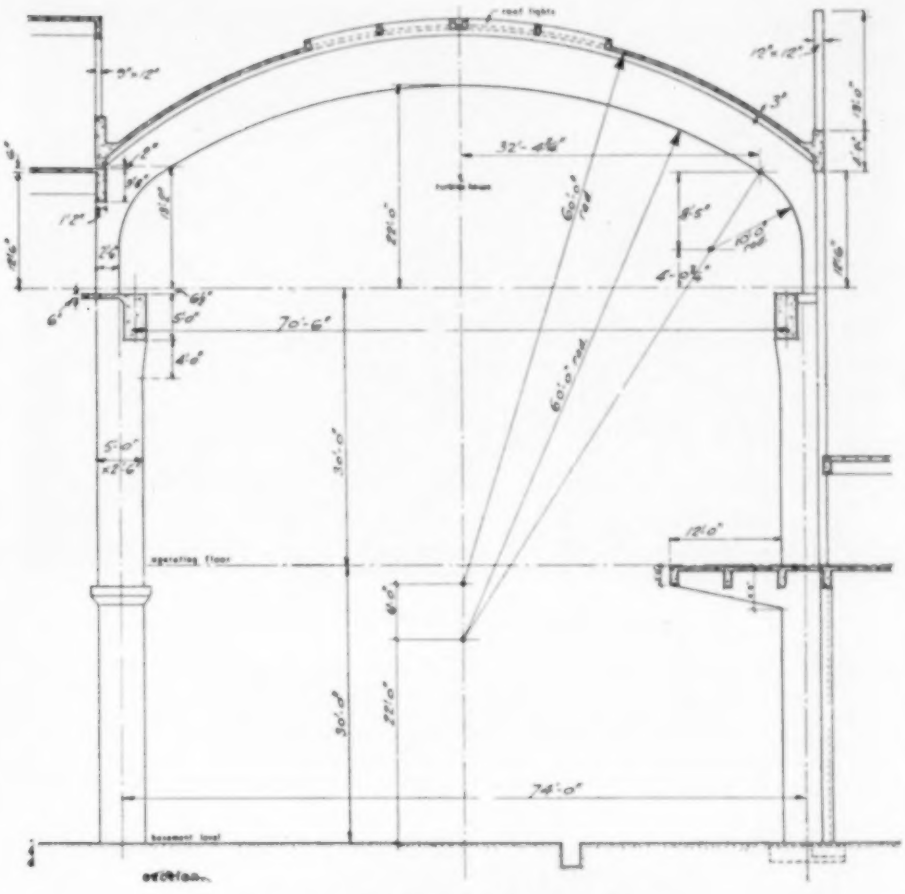


Fig. 2.—Cross Section of Turbine House.

adjoining steel-framed boiler house are also resisted by the concrete frame. The dimensions and arrangement of the reinforcement are shown in *Figs.* 2 and 3.

Construction of Roof.

The height above ground to the crown of the roof is 88 ft. Therefore the roof was constructed from a carriage travelling on the crane rails, a method that left the floor clear of scaffolding except that required for the construction of the columns, thereby permitting other work to proceed unhampered. The carriage (*Fig.* 4) comprised two main trusses 32 ft. apart and spanning across the building. The trusses were braced together by steel joists which carried working platforms and a light steel frame supporting the roof centering for one bay. The main trusses passed below the ribs, and when travelling it was necessary only to lower the roof centering and supporting frame on to the platform. The carriage was then pushed forward to the next bay. The entire operation could be done in eight hours. Ten days were allowed for the concrete to harden before removing the centering, but a transverse tie was inserted between the main columns just below the springings to reduce the stresses after removal. The tie comprised a ¾-in.

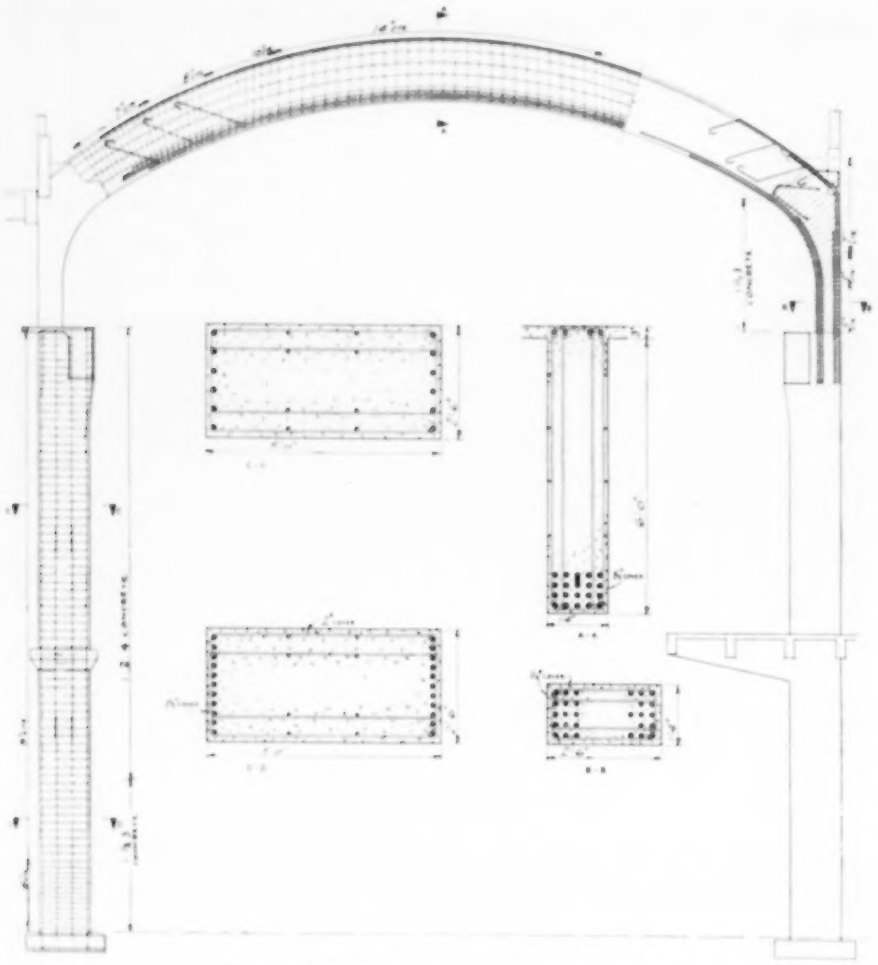


Fig. 3.—Reinforcement in Portal Frames.

plate 8 in. wide connected by turnbuckles to a stirrup around the column, and supported by cradles on the cross members of the carriage. Since the columns are 31 ft. apart and the trusses of the carriage were 32 ft. apart the carriage could be moved without interfering with the tie. On completion of a bay the tie was attached to the two forward columns, which were just behind the forward truss of the carriage, and when the carriage was moved to the next bay the tie remained in position and was then just in front of the rear truss. The tie could therefore remain in position until the adjacent bay was constructed and enabled the concrete to mature for an additional 2½ weeks before the full dead-load stresses were induced in the frame.

Expansion Joints.

Expansion joints (*Figs.* 5 and 6) are provided in the roof in the middle of every third bay, where double secondary ribs are provided; this arrangement is simpler than the use of double main ribs and columns at expansion joints between bays.

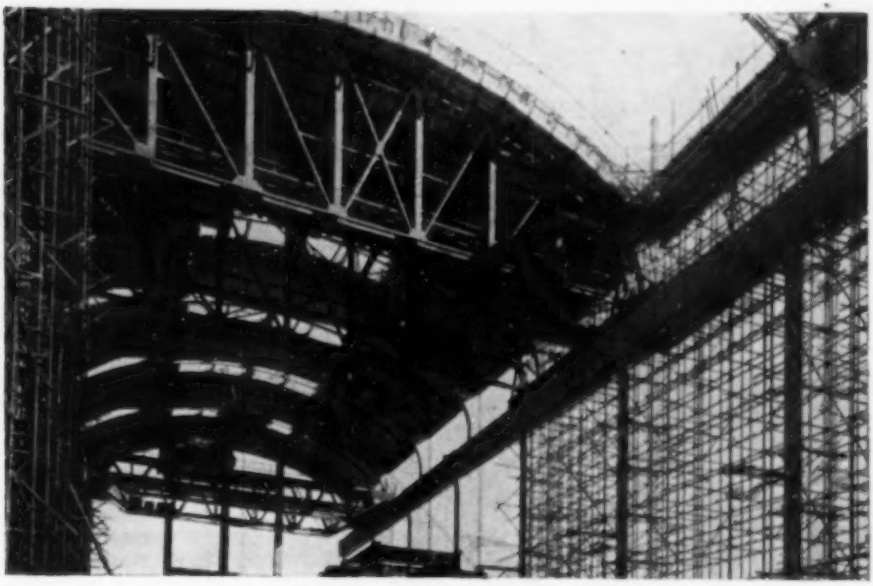


Fig. 4.—Travelling Centering for Roof.

The two cantilevered parts of the roof slab act temporarily as a complete member spanning a distance of 31 ft. across the bay, the temporary connection across the gap of the expansion joint comprising steel bars passing through a precast block. Each connection had to transmit the longitudinal forces in the slab. When sufficient bays had been constructed for the half-bays to act as cantilevers, the concrete blocks and bars were cut. Because of the large forces acting across

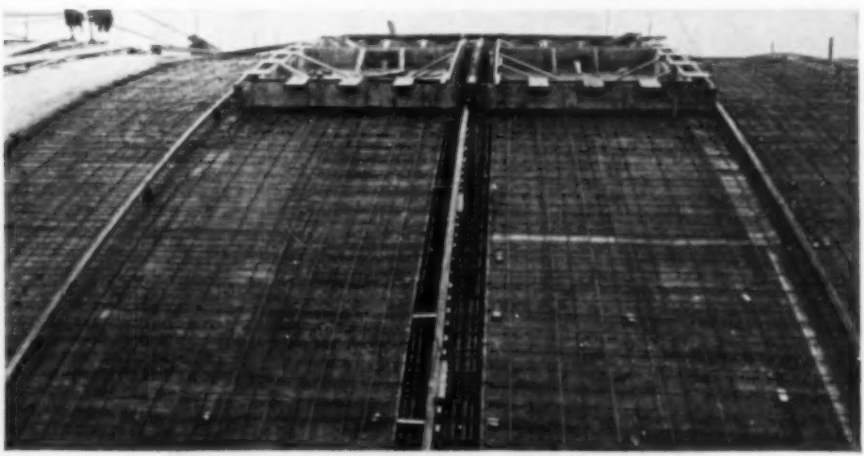


Fig. 5.—Expansion Joint in Roof.

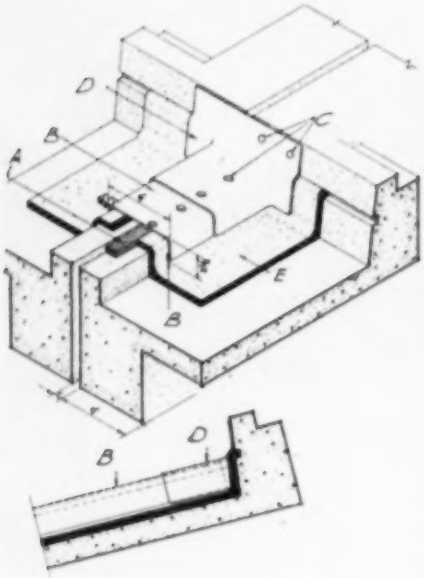


Fig. 6.—Detail of Expansion Joint in Roof.

the connections, it was necessary to cut out the temporary concrete before severing the bars in order to avoid spalling the permanent concrete.

The expansion joints are covered as shown in *Fig.* 6, where A is a 1½-in. by ¾-in. hardwood batten fixed by countersunk screws to dovetailed wooden plugs at 2 ft. centres. The ¾-in. layer of asphalt E is laid up to batten A on one side of the joint. A capping strip B of No. 24 gauge copper is laid over the joint and secured to A by copper nails. The lead flashing apron D is similarly fixed by the copper nails C to the batten and to wooden plugs let into the concrete.

The mixture of the concrete is generally 1 : 1½ : 3 except for the part of the main columns between 15 ft. and 60 ft. above the foundations where 1 : 2 : 4 concrete is provided. The working compressive stress in the 1 : 1½ : 3 concrete is 1100 lb. per square inch. The columns were cast in lifts of 8 ft. and construction joints (*Fig.* 7) were formed in the ribs just above the springings. The columns and edge-beams at the springings were therefore constructed in advance of the roof. The

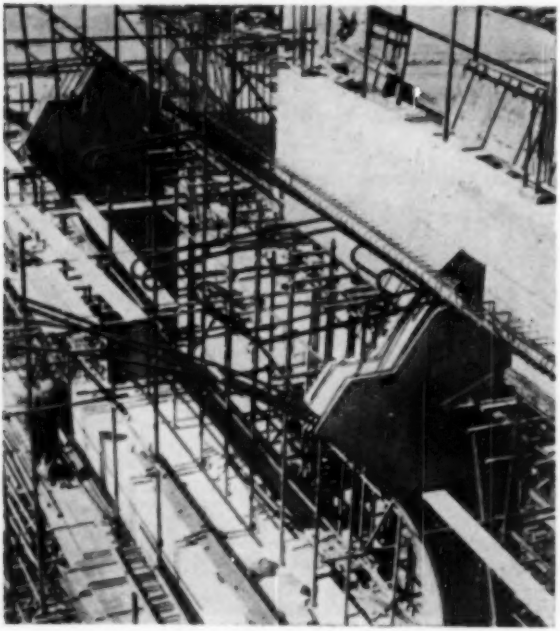


Fig. 7.—Construction Joint at Springing.

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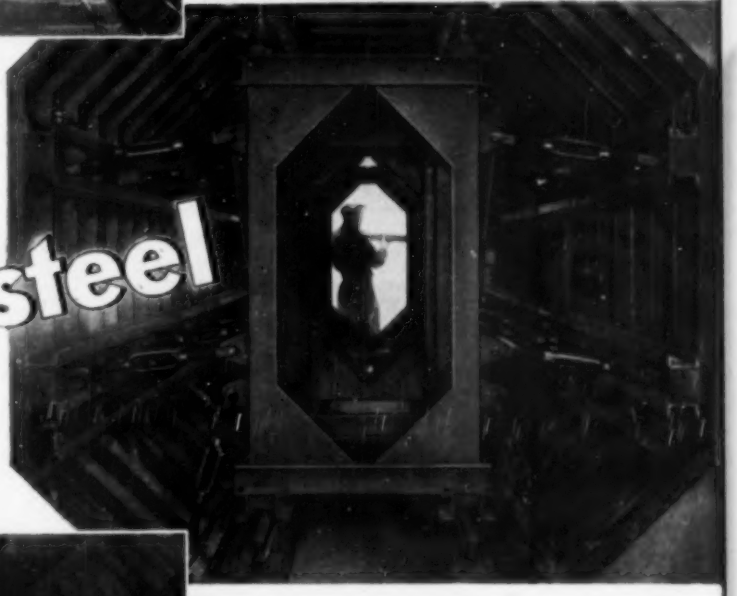

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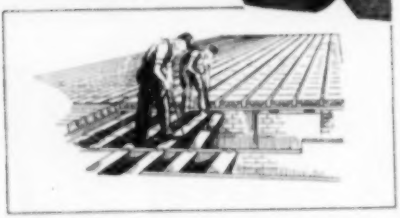

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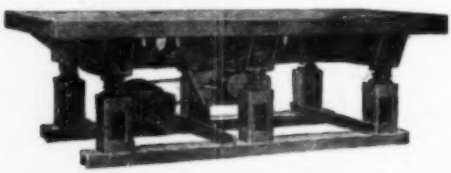

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slab of the vault is 3 in. thick and was placed between screeds at 8 ft. centres following the curve of the vault (*Fig.* 5).

An opening 32 ft. by 15 ft. occurs in each bay of the roof and is divided by ribs supporting eight precast panels of roof lights. The tops of the kerbs supporting the roof-lights are slotted at intervals to provide ventilation sufficient to prevent condensation on the underside of the glass. The roof is covered externally with two layers of mastic asphalt and will be sprayed internally with ¾ in. of asbestos to provide thermal insulation.

The rate of constructing the turbine house was about one bay each month. The architect for the power station is Mr. R. A. H. Livett, O.B.E., A.R.I.B.A., City Architect of Leeds. The consulting engineers to the British Electricity Authority are Messrs. Merz & McLellan and Sir Alexander Gibb & Partners. The contractors for the turbine house are Messrs. Holst & Co., Ltd., who designed the reinforced concrete work in collaboration with Chisarc & Shell 'D', Ltd., to whom Mr. H. G. Cousins, B.Sc., M.Inst.C.E., was consulting engineer.

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A New Prestressed Concrete Sleeper.

In order to reduce the weight of prestressed concrete sleepers, and to avoid the possibility of plain wires slipping under fluctuating loads, a light sleeper with the section at the middle reduced in size has been designed in France by M. Freyssinet. The first of these sleepers weighed only 290 lb., was prestressed by 42 crimped wires of 0·1 in. diameter, and the thickness of the part under the rail was 4·8 in. They were manufactured on long production lines and tried in secondary tracks. From these trials it was found that for main lines a larger sleeper exerted. The prestressing force acts centrally in the middle part of the sleeper and eccentrically in the parts under the rails. Experience has shown that for the sleeper to withstand the prescribed test the concrete must have a compressive strength of 7,000 lb. per square inch after 28 days. The length of the sleeper is 7 ft. 9 in., the width 9⅜ in., and the volume of concrete is 2·15 cu. ft. The thickness of the middle part is 3¼ in. The weight of high-tensile wire in one sleeper is 11½ lb. and there are 1½ lb. of mild steel secondary reinforcement.

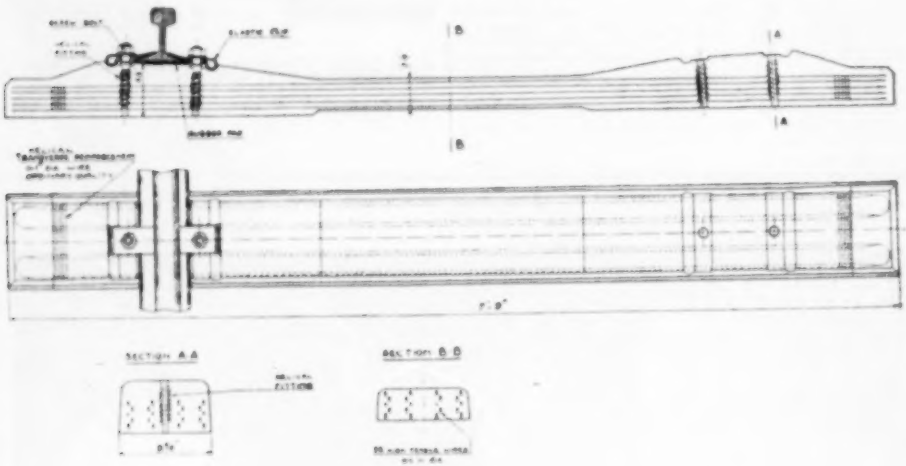

Fig. 1.

was required, the principal test of which should be carried out after delivery and should be a bending test in which the load is applied at one end of the sleeper through the centre-line of the rail. The load should be distributed along a length of 5·3 in. and two supports should be placed under the sleeper 7·9 in. on either side of the centre-line of the load. A load of 30 tons in this condition produces a bending moment of 13,300 ft.-lb. and must not cause cracks.

A new type of sleeper (*Fig.* 1) has been designed by M. Freyssinet to comply with the foregoing test. The thickness of the sleeper under the rail is 5½ in. and the sleeper is prestressed by 56 crimped wires of 0·1 in. diameter which enable an initial stretching force of 44 tons to be

The reduced middle part of the sleeper is an essential feature of the design. In some types of concrete sleepers the middle part is relatively thick and has to resist the bending moments produced by the loads. The bending moment can be increased by abnormal reaction from the ballast under the middle of the sleeper or by the rails not being at the correct levels. The new sleeper acts almost like two blocks connected by a tie. The middle part can deflect 0·4 in. without cracking. The reduction in thickness of the middle part and the general reduction in size of the sleeper made possible by the high strength of the concrete, and the use of crimped wires of which the bond is more certain, produce a main-line sleeper which weighs only 340 lb. and which, although

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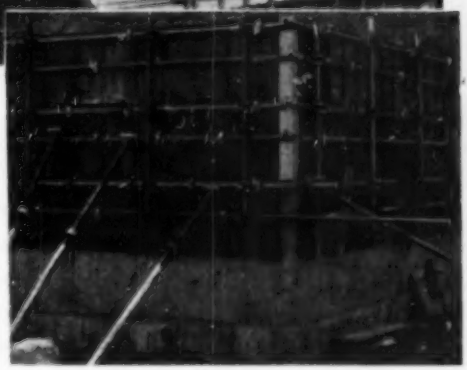

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heavier than a wooden sleeper, can be lifted and placed in the track by hand.

To avoid contact between the ballast and the middle of the sleeper, it is usual on French railways to leave a channel in the ballast in the middle of the track; this method also saves ballast.

A feature of the design is the method of obtaining a positive anchorage for the prestressing wires without using any anchorage device which must be left in the concrete. The wires are placed in two skeins, the ends of which project from the two ends of the sleeper when it is cast since a length of 4 in. is omitted at each end of the sleeper when first cast. The two 4-in. blocks are cast at a subsequent stage in the manufacture, and protect the loops at the ends of the wires from corrosion. An advantage of casting the end blocks after the stretching force has been transferred to the concrete is that there is no difference in the stresses in the concrete inside and outside the wire loops at the ends of the sleeper, as is the case when the force is transferred to the concrete after casting the entire sleeper. In course of time the concrete deforms plastically and sufficient precompression is induced in the end blocks to ensure that the sleeper is strong enough at its more lightly loaded ends.

The rail bears on a grooved rubber pad and is held down by two clips of spring steel. The clips are attached to the sleeper by bolts screwed into a helical wire fitting, sealed with bituminous mastic, and provided with a nut and washer. In a severe dynamic test carried out by engineers of the French railways, after 100 hours on a vibrating machine there was no sign of fatigue in the sleeper or the rail attachment, nor was there any loosening of the bolts. In a similar test carried out on a new oak sleeper with the type of rail attachment standard on French railways, the rail penetrated $\frac{3}{8}$ in. into the sleeper.

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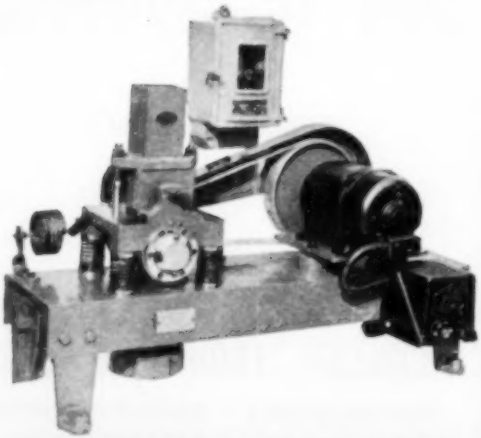

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It is stated that sodium-silicate cements, which are porous, can withstand temperatures up to 1600 deg. F. and are resistant to strong acids (except hydrofluoric), but that they are unsuitable for weak acids and alkaline solutions. These cements are of two kinds, namely, (1) a mixture of crushed silica (80-mesh to 325-mesh, U.S.A. sieves) and sodium-silicate, the product setting by loss of moisture, and (2) quick-setting sodium-silicate cement which depends on the chemical reaction of a hardening agent with sodium silicate, the agent being dispersed in a crushed silica filler which is mixed with sodium-silicate at the time of use.

Sulphur cements, which are composed of sulphur and crushed silica or carbon, can be made of varying plasticity by the addition of organic sulphides, are suitable for fluctuating temperatures up to 200 deg. F., are resistant to most strong and weak acids (except chromic acid, and nitric acid over 40 per cent. concentration), water, and weak (but not strong) alkaline solutions; of low porosity; soluble in some organic solvents; have a tensile strength of 600 lb. per square inch and a compressive strength of 6000 lb. per square inch; and set within a few minutes even in frost. The principal use is for mortar for acid-resistant brickwork. Tanks containing a steel pickling mixture of nitric and hydrofluoric acids at 160 deg. F. have been lined with carbon bricks set in sulphur-carbon cement.

Phenolic-resin cements, which are also used for acid-resistant brickwork, are resistant to most acids that are not highly oxidizing, solvents, weak alkalis,

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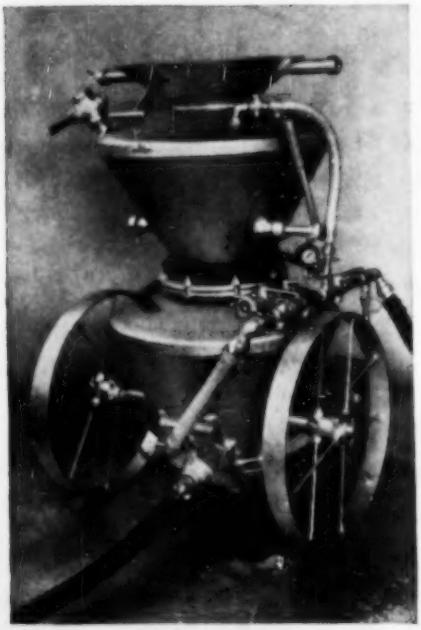

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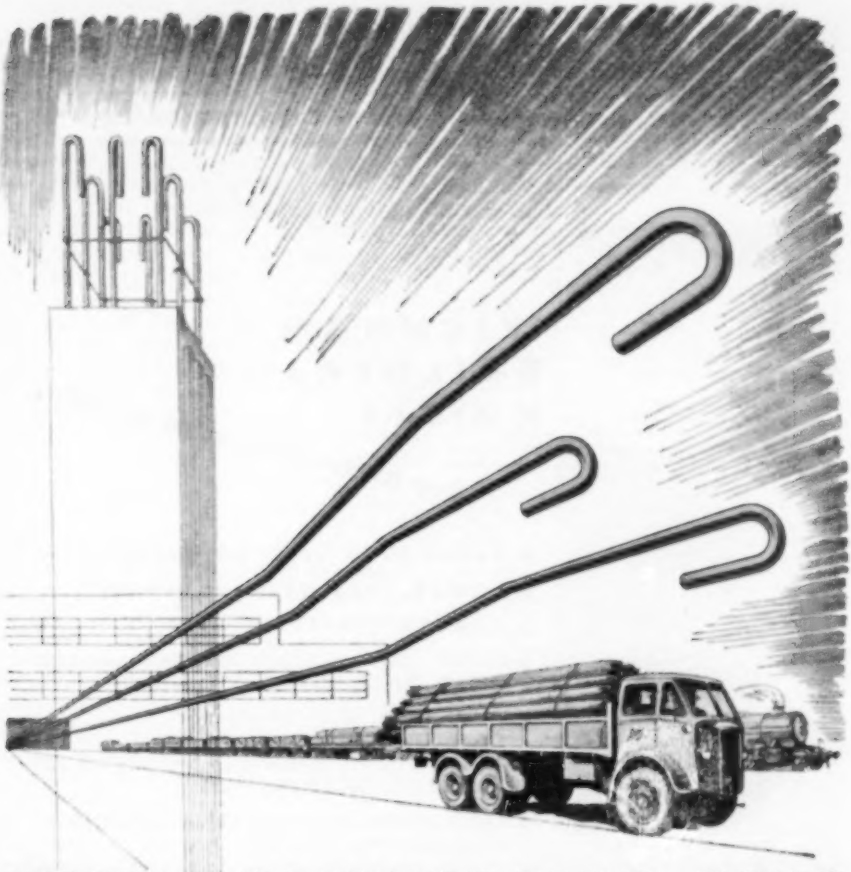

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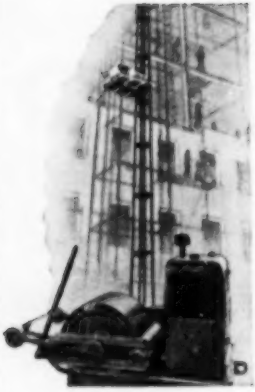

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steam at 360 deg. F., fats, and oils, have a tensile strength of 1000 lb. per square inch and a compressive strength of 10,000 lb. per square inch, and are of low porosity. Liquid phenolic resin containing about 75 per cent. of solids is mixed at the time of using with a silica or carbon filler containing a hardening agent, which produces controlled but rapid setting of the liquid resin at atmospheric temperature.

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MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s.; vacancies advertised are restricted to persons or employments excepted from the provisions of the Control of Engagement Order, 1947. Other miscellaneous advertisements 4d. a word, 10s. minimum. Box number 1s. extra.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT

SITUATION VACANT. Experienced reinforced concrete designer-detailer required by civil engineering contractors in their London head office. Reply giving details of experience, age and salary required. Box 2341, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. London firm of reinforced concrete engineers and contractors require engineer, preferably B.Sc., A.M.I.C.E. and/or A.M.I.Struct.E., to take charge of their Birmingham design office. Extensive experience in design of all types of reinforced concrete structures essential. Write giving full particulars of qualifications, age and past employment to MANAGING DIRECTOR, Box C.C.551, at 191 Gresham House, London, E.C.2.

SITUATION VACANT. Reinforced concrete engineer required to specialise in the design of reinforced concrete floors, roofs and staircases. The vacancy is in London head office. Write giving full details of experience, and state salary required to Box C.C.548, at 191 Gresham House, London, E.C.2.

SITUATION VACANT. General foreman required for reinforced concrete silos. Good opportunity for man with first-class ability and experience. Write age, full particulars and salary required. Box 2337, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Designer. A vacancy occurs with large engineering contractors in the North-West, specialising in complete erection of electricity power stations, for a senior reinforced concrete and/or structural steelwork designer. Substantial salary. Pension scheme. Write stating age, qualifications, experience and present salary. Box 2338, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Draughtsman required for pre-cast factory near Dudley. Structural steel experience useful. Pensionable appointment. TARSLAG, LTD., 117 Dunstall Road, Wolverhampton.

SITUATIONS VACANT. Consulting engineer in London has vacancies for two reinforced concrete detailers. Salary £650 and £750 respectively. Applications, stating full particulars, to Box 2347, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. Consulting engineer in Westminster has vacancies on permanent staff for two quantity surveyors. Commencing salaries £550 and £450 respectively. Applicants must have had considerable experience in measuring reinforced concrete work but knowledge of general building trades not essential. Applications will be considered from men who have gained such experience while working with contractors or reinforced concrete specialists. Experience in making interim measurements of work and final accounts is an advantage. Box 2348, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Experienced reinforced concrete designer, preferably B.Sc. and/or A.M.I.C.E., required for Manchester office of a London firm of reinforced concrete engineers and contractors. Write, giving full details of experience and age, to the MANAGING DIRECTOR, Box C.C. 599, at 191 Gresham House, London, E.C.2.

SITUATIONS VACANT. Experienced reinforced concrete designers and detailers required in London, W.1, office. Five-day week. Canteen. Write, stating experience, etc., to "TWISTEEL" REINFORCEMENT, LTD., 43 Upper Grosvenor Street, W.1.

(*Continued on page 72.*)

MISCELLANEOUS ADVERTISEMENTS

(*Continued from page 71.*)

SITUATIONS VACANT. Applications are invited by consulting engineers, Westminster, for the following vacancies: (1) Detailer-designer, £900 per annum. (2) Detailer, £700 to £750 per annum. (3) Junior detailer, £550 to £650 according to experience. Applicants must be thoroughly experienced in work in reinforced concrete specialists' office. Applications giving age and details of experience will be treated as confidential. Box 2352, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Reinforced concrete designer-draughtsman with wide competitive experience required for Manchester office of reinforced concrete engineers. Pension scheme in operation. Capacity for supervision of juniors necessary. State age, experience, and salary required. Box 2355, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Reinforced concrete engineers require for London office an experienced taker-off. Write full details, experience, age and salary required. Box 2356, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. Structural draughtsman wanted for consulting engineer's office. Capable of detailing reinforced concrete structures, and with good drawing-office experience. Apply in writing, stating age, experience and salary required, to F. J. SAMUELY, 29 St. Giles High Street, London, W.C.2.

SITUATION VACANT. Experienced senior reinforced concrete designer required who is familiar with L.C.C. requirements and can undertake preliminary design and own production of detail drawings. Some knowledge of civil engineering and structural steelwork desirable. Varied interesting work in all engineering branches. London, West-End area. Good prospects and salary with additional incentive bonus. Advertiser's staff has been notified of this advertisement. Write, stating age, experience and salary required, to Box C.C. 626, at 191 Gresham House, London, E.C.2.

SITUATION VACANT. Reinforced concrete checker-designer required. Experienced in foundations and concrete structures for heavy equipment for oil refineries. Address application in first place by letter to the attention of CHIEF DRAUGHTSMAN, THE LUMMUS CO., LTD., 525 Oxford Street, London, W.1, stating age, experience, salary expected and when free to commence.

SITUATION VACANT. Draughtsman with at least three years' experience in reinforced concrete detailing required immediately. Apply in writing to PETER LIND & CO., LTD., Stratton House, Piccadilly, London, W.1, stating age, experience and salary required.

SITUATION VACANT. Designer-draughtsman experienced in reinforced concrete for buildings, structures, and foundations for equipment. Pension scheme, five-day week. Write in confidence, stating age, experience and salary required, to CHIEF ENGINEER, E. B. BADGER & SONS (G.B.), LTD., 40 Parkgate Road, London, S.W.11.

SITUATION VACANT. Estimator required for modern and rapidly expanding prestressed and precast concrete manufacturer. Only applicants with comprehensive knowledge of the industry considered. Congenial appointment offered to man with real ability. Write in first instance to the MANAGER, H. B. CONCRETE CO., LTD., East Street, Epsom.

MINISTRY OF WORKS.

Drawing office staff are required in the Structural Engineering Division, for duty in London, Manchester, Risley (Warrington), and Tunbridge Wells. Hostel accommodation is available at Risley. Salary in London: £300 at age 21 with £20 additional for each year above up to maximum of £525. Rates outside London are slightly lower. There are posts of leading structural engineering assistants, salary £500–£625 per annum (London), which are filled by promotion. Applicants should be experienced in design and/or detailing of either reinforced concrete or structural steelwork. The work is varied and not confined to standard schemes.

Applications would also be considered for drawing office assistants with suitable technical training and no experience, age between 18 and 21, salary £175 per annum rising to £250 per annum. Although these are not established posts, some of them have long-term possibilities, and competitions are held periodically to fill established vacancies.

Apply in writing, stating age, nationality, full details of experience, and locality preferred, to CHIEF STRUCTURAL ENGINEER, W.G.10/AP, MINISTRY OF WORKS, Abell House, John Islip Street, London, S.W.1.

METROPOLITAN BOROUGH OF PADDINGTON.
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Applications are invited for the appointment of a resident engineer on the temporary establishment of the Housing Department, who will be required to supervise the construction of multi-story flats in reinforced concrete on the above-mentioned housing site. The salary will be between £550 and £700 per annum, dependent upon the qualifications and experience of the successful candidate. Previous experience in reinforced concrete work is essential, and preference will be given to candidates who are Associate Members of the Institution of Structural Engineers and/or the Institution of Civil Engineers. The appointment will be subject to one month's notice on either side, the provisions of the Council's Superannuation Acts and Standing Orders, and to the National Joint Council's Scheme of Conditions of Service. Applications for this appointment, stating age, qualifications, present and previous appointments with salaries and experience, together with the names of three authorities or persons to whom reference may be made, should be delivered to the undersigned, in an envelope endorsed "Application for the Appointment of Resident Engineer", not later than noon on 17 February, 1950. No forms of application will be issued. Candidates must state, in writing, whether, to their knowledge, they are related to any member or senior officer of the Council. Canvassing directly or indirectly will disqualify.

W. H. BENTLEY, Town Clerk.

Town Hall, Paddington, W.2.
2 February, 1950.

SITUATIONS WANTED

SITUATION WANTED. Qualified reinforced concrete engineer, recently arrived back in the U.K., with over eight years' experience in design, construction, estimating, etc., of reinforced concrete structures, seeks a post as reinforced concrete designer in a London office. Willing to do detailing if required. £600 p.a. Box 2359, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION WANTED. Chartered structural and civil engineers (5) with considerable experience in reinforced concrete and structural steelwork design and detailing are available for part-time work. Box 2362, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SERVICES OFFERED. Fully qualified and experienced structural engineer offers part-time services to consulting engineers for the design and detailing of all types of reinforced concrete structures. Write in first instance to Box 2360, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION WANTED. Draughtsman-designer, reinforced concrete, will undertake spare-time outwork at home. Few hours weekly. Experience in precast and in situ work. Write giving full details. Box 2357, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

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Published by CONCRETE PUBLICATIONS LTD., 14 Dartmouth Street, London, S.W.1. Telephone: WHITEHALL 4581
Printed by BUTLER & TANNER LTD., The Selwood Printing Works, Frome and London